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20 April 1990

Mr. Holland Shepherd, Reclamation Soils Specialist
Mined Land Reclamation Program
State of Utah Natural Resources
Department of Oil, Gas and Mining
3 Triad Center, Suite 350
Salt Lake City, Utah 84160-1203

Dear Mr. Shepherd:

Enclosed please find a copy of our Lisbon operation reclamation plan, as submitted to the Nuclear Regulatory Commission in 1985.

Several design changes have taken place since that time involving detailed engineering studies and I expect there will be more changes before final approval is obtained from NRC, which I hope will be this year. The most significant change to-date has been the dropping of the slurry idea for tailings cover placement.

If you have any questions, or need additional information, please call me at (801) 686-2216.

Sincerely,

R. S. Pattison
Manager

RSP:tw

Lisbon Mine Operations, Lasal Route, Moab, Utah 84532 • (801) 686-2216 • FAX (801) 686-2337

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5.5.12 Detailed Reclamation Plan

With today's depressed yellowcake value, RAMC has no plan to construct an acid circuit. A detailed reclamation plan for the long-term isolation of only alkaline leach tailings shall therefore be addressed in this Section. Also, RAMC has no current plan to shut down the mill and reclaim the tailings impoundments, as was suggested in August 1984 (see Table 5.5-7). Isolation of the largest foreseeable tailings volume shall be addressed: that which would result if the mill ran continuously, at full production, until the end of the current license period--30 September 1989. However, if RAMC decided to proceed with either of the above-mentioned plans before that date, this reclamation plan shall be revised accordingly and submitted to the U.S. NRC, Uranium Recovery Field Office, for review and approval in the form of a license amendment.

Pre-Reclamation Tailings Management

Final abandonment contouring is designed to make the best use of geologic and existing topographic features. The design provides for the long-term isolation of tailings and is intended to meet all regulatory requirements at a minimum cost. Recontouring of tailings to conform to final abandonment requirements is considered extremely difficult, if not impossible; heavy equipment easily "bogs down" in the thinnest layers of this finely-ground, high-moisture-content "slimes" tailing material. Therefore, one of the main objectives in tailings management is to contour tailings slopes during their initial deposition so that they conform to the final reclamation contours, thus avoiding the need for any substantial amount of recontouring.

Tailing disposal is also managed to prevent unnecessary dusting of beaches, ponding, and seepage to the best extent practicable. Details of interim tailings dust stabilization are given in Section 5.5.7. To eliminate unnecessary ponding, tailings are generally deposited to slope with the natural valley topography in the impoundment area; that is, the discharge point is repeatedly moved uphill and farther from the embankments. This disposal system affords substantial economy in that it permits large quantities of tailings to be deposited above embankment crest elevations, thus reducing the embankment height that would otherwise be necessary. It also permits a minimum ponded area and increases flood storage capacity. In fact, if the method had been used earlier in the life of the operation, the upper tailings embankment may not have been needed.

Because no seepage barrier is provided beneath the existing tailings area, all unnecessary ponding is avoided, especially against bare earth. Tailings discharge lines are therefore provided around most of the impoundment's perimeter. At the first sign of ponding, the line is broken at that point so that tailings solids can eliminate the ponding. In addition, again to reduce seepage, 100-foot beaches are maintained around the pond perimeter that is not already protected by the main beach slope.

At its present embankment crest elevation, the upper tailings impoundment has essentially been filled to capacity with tailings. A new sprinkling system has been installed to provide almost 100% coverage of its beaches and to prevent blowing tailings prior to reclamation of this impoundment. No additional tailings will be run into the upper impoundment unless demanded by an unforeseen emergency. A triple 65" x 40" culvert outflow structure will pass major floods to the lower impoundment, thus preventing embankment overtopping before this reclamation plan is implemented.

All tailings have been deposited into the lower impoundment since late January 1985. The method of deposition is designed to minimize blowing tailings during operation. Tailings are generally deposited along a parallel strike to the upper embankment alignment to create a 2% slope down and away from the embankment downstream face. This method provides several advantages: it reduces the riprap needed for protection of the downstream face after abandonment; it creates the best slopes for sheet rainfall runoff; it allows an optimum tailings cover design in that the excess lower tailings embankment material can be used for covering the west-end lower impoundment tailings, affording flatter embankment slopes with a minimum need for riprap; and, it gradually eliminates the upper embankment as a structure to be protected. A minimum pond elevation is maintained to minimize the seepage driving force, and 100-foot beaches are maintained around the remaining pond perimeter. Except for the occasional rain storms and snow melt, the lower pond elevation will remain below 6,638 feet msl for the license period. The pond surface area will be reduced as the main tailings beach advances westward.

Seepage Control

Regional stratigraphy and natural groundwater regimes are discussed in Section 2.3.1 and site-specific groundwater seepage control is described in Section 4.2.2. Results of additional groundwater investigations conducted at the Lisbon mill are detailed in the referenced documents at the end of this Section. Migration of contaminants from the Lisbon mill was found to be largely controlled by fractures in the Dakota-Burro Canyon sandstone formation. A fracture, trending north 60° west from the northwest corner of the upper tailings impoundment was intercepted at well #OW-UT9 during the field test drilling program and was found to be the major cause of contaminant leakage from the tailings impoundments. The well is located near the upper impoundment tailings and the fracture was found to provide a conduit for rapid migration of contaminants from the upper tailings impoundment to the north property boundary dear monitoring well #MW7. A pumpback system was therefore installed to preclude further migration of contaminants in the fracture beyond the pumping well (OW-UT9). Seepage not intercepted by the recovery system is small and is diluted to harmless concentrations when it reaches natural groundwaters.

The existing recovery well system will be operated continuously as long as there is contaminated solution to pump. Prior to mill shutdown, this solution will be returned to the lower tailings pond rather than the upper. Also, other mill solutions will be diverted to the lower pond in order to help dry-out the upper pond and its beaches. Rainfall runoff will tend to reduce contaminant levels in the upper pond. This "clean" water will therefore be diverted to sprinkle the lower impoundment beaches after mixing with more contaminated solutions in its pond to prevent dissolving the beneficial precipitant "crust" from the lower impoundment beaches.

Solution in the lower tailings pond will be at its normal contamination levels at the time of mill closure. After closure, the solution will be allowed to dry out. Drying will be enhanced by continuously using a spray field in the lower impoundment. The lower impoundment can be dried up while placement of reclamation cover is underway in the upper impoundment. By the time reclamation of the upper impoundment is complete (about one year), the lower impoundment will be ready for covering. In addition, seepage recovered by the pumpback system is expected to clean-up following clean-up of the upper tailings pond water.

The tailings consist of more than 80% minus 325 mesh particles and tend to retain high moisture contents as perched water tables for indefinitely long time periods. Notwithstanding the dry climate and the high evaporation rates in the region, only the top inch or two is actually dried out. Below this depth the tailings retain their initial moisture content, which is normally more than 20%. This fact implies that the in-place tailings are relatively impervious. In addition, it is thought that the upper tailings pond itself is feeding the major fracture described above. If so, seepage from well #OW-UT9 could clean up quickly. However, pumpback will be continued and an evaporative spray field shall be maintained as long as necessary; the length of time needed cannot be predicted with certainty.

Cover Design and Construction

Estimates of cover materials required to meet NRC's 2 pCi/m^2-sec standard are presented in Section 5.5.10. The field tests undertaken to arrive at those estimates are not too reliable for two reasons: inadequate attention was paid to compaction of the tailing and soil cover samples to simulate in situ conditions; and, both the tailing and soil sample moisture conditions were too low to be representative of the real situation. Both reasons give rise to unreasonably high radon flux exhalations. Furthermore, all tests were conducted on old tailing material from the lower impoundment which was the waste byproduct from ores grading as high as 0.40% U_3O_8. Today's grades are closer to 0.20%.

Criterion 6 of NRC's modified uranium mill tailing regulations in the amended Appendix A of 10 CFR, Part 40, requires that sufficient earthen cover be placed over tailings at the end of operations to provide reasonable assurance that the average radon-222 release rate to the atmosphere does not exceed 20 pCi/m^2-sec. The cover shall be effective for 1,000 years to the extent reasonably achievable and direct gamma exposure should be reduced to background levels.

RAMC plans to provide an earthen cover which complies with the requirements of Criterion 6. No near surface cover materials such as mine waste rock, that contains elevated levels of radium, or non-soil materials will be used as cover materials. Soil used for near surface cover shall be essentially the same, as far as radioactivity is concerned, as that of surrounding surface soils to ensure that surface radon exhalation is not significantly above background because of the cover material itself.

A depth profile of natural soil moisture content was taken from the proposed borrow area for cover material. A front-end loader was used to dig a five foot deep trench and samples of soil were immediately collected from one side of the trench at one foot intervals and analyzed for moisture content. The results indicated that the moisture content does not vary much with increasing depth, averaging seven percent by weight moisture down to five feet. The surface two inches of soil showed a low moisture content (~2%) due to solar drying. Depth profiles were also taken of in situ tailings moisture down to five feet and the top one-inch layer was sampled separately because it had dried out in the sun. The results also showed a relatively even moisture distribution below one-inch, averaging 20.7% by weight. Surface moisture averaged 7.7%. Due to the importance of the effect of moisture content on the tailings diffusion coefficient, and its consequent effect on attenuation of radon flux and cover thickness needed, the matter was investigated further.

Data presented by Ford, Bacon and Davis (1981) and Mountain States Research and Development (1982) for abandoned alkaline-leach tailings piles near Ambrosia Lake, New Mexico, and Tuba City, Arizona, suggest that the tailings will continually have a high moisture content. FB&D (1981) collected composite samples from two drill holes at Ambrosia Lake in 1976 that had moisture contents of 22.7% (to 8.5 foot depth) and 21.8% (to 9.5 foot depth), thirteen years after plant shutdown. MSR&D (1982) similarly found an average moisture content of 20.7% at the Tuba City site, fifteen years after plant shutdown. Hence, the average value of 20.7% at the RAMC site can be considered representative of future conditions. Further, after remaining ponded areas (the upper and lower tailings ponds) have dried out RAMC believes that due to the tailings fineness, moisture contents will remain high for an indefinite period of time--perhaps up to 1,000 years. A typical tailings gradation is presented in Figure 5.5-1.

Although reclamation of an inactive tailings impoundment primarily involves placing an adequate cover over the abandoned tailings, the specific implementation program is dependent on the specific nature of the tailings pile to be reclaimed. Reclaiming a dry, coarse-grained pile is not a difficult proposition, and performance of the cover is relatively predictable. However, placing a cover over slimes tailings such as RAMC's with its perched water tables and high water content by conventional earth-moving methods is expected to be difficult, if not impossible, as has already been mentioned. Attempts to place covers on wet, saturated tailings results in displacement of the tailings, loss of cover materials into the tailings, and probable loss of expensive equipment. The method of waiting long enough for a competent crust

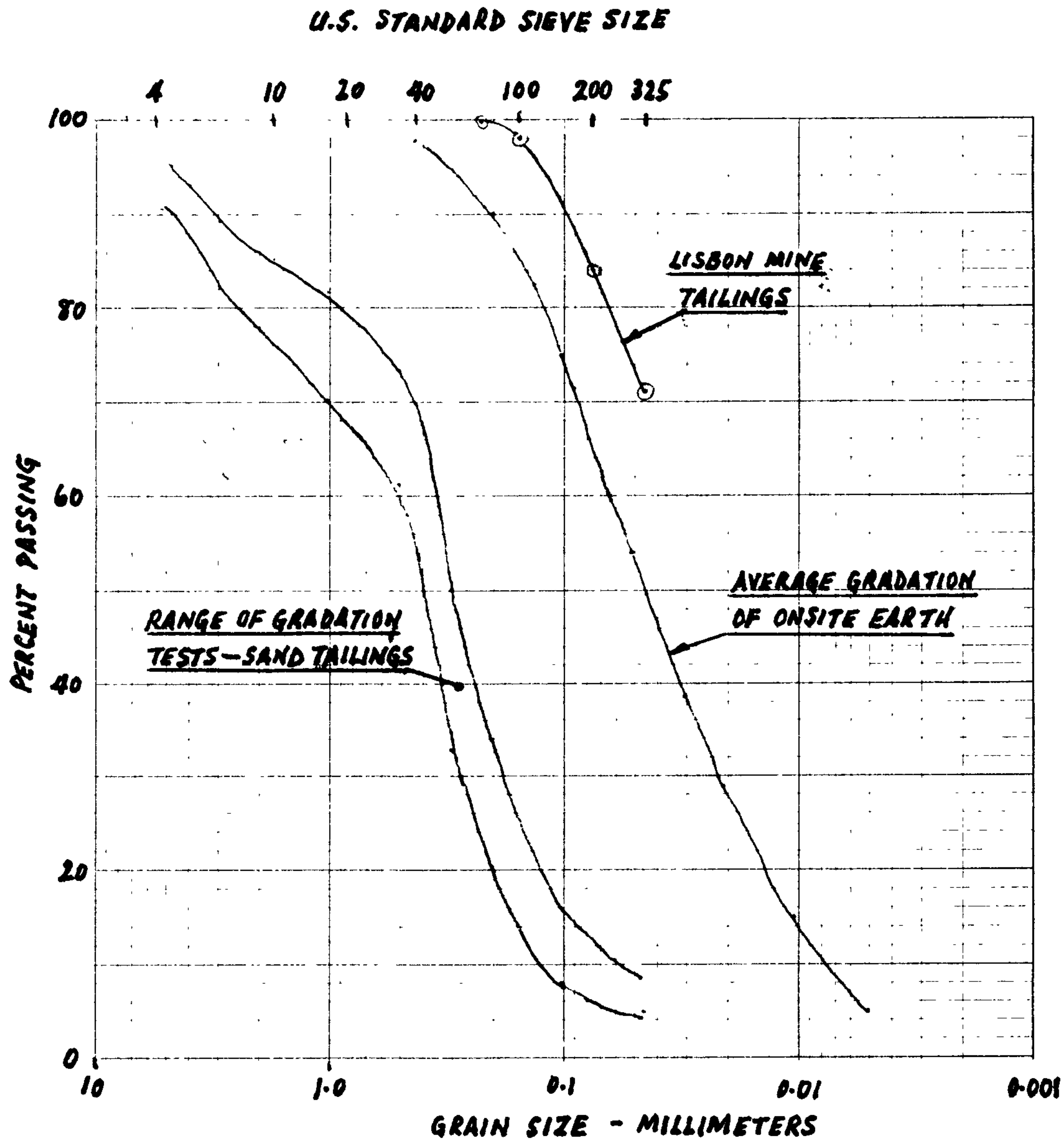


FIGURE 5.5-1 TYPICAL GRADATION DATA

to form by surface evaporation was considered, but was rejected by RAMC. It will take too long, and therefore not meet the intention of Criterion 6.

Other methods such as installation of drains beneath or through the tailings to facilitate drainage and accelerate stabilization through lowering the water table and consolidation may be considered because pumpback well #OW-UT9 appears to be effectively working this way. However, there is no indication yet that the water table is being lowered by the well, and it has run at 30 gpm for almost two years now; it is suspected that the well is in direct connection with surface ponded water. Besides, it is too late now to implement such a system of drainage because there are more than 3×10^6 MT of tailings already in place.

Methods such as consolidation by dynamic compaction, electro-osmosis and other sophisticated techniques have been used elsewhere. RAMC has no experience of those methods, but will investigate to see if any can be usefully applied at Lisbon. In the interim, with no acceptable effective plan--that is, one which will result in even consolidation in a reasonable period of time--available to dewater its tailings piles, RAMC intends to reclaim them in their current condition; perhaps, further consolidation is unnecessary.

Cover thickness can now be determined. The calculation of the thickness of cover materials required to attenuate radon flux to near-background levels is generally based upon diffusion theory. NRC uses this theory in Appendix P of their Generic Environmental Impact Statement on uranium milling (GEIS) and, after making several assumptions, provide convenient tables of required depths of cover for radon attenuation to meet 2pCi/m^2-sec criteria. For a particular grade of ore it is only necessary to know the expected long-term moisture contents of the tailings and the cover material to find the depth of cover needed to achieve the $2 pCi/m^2$-sec standard. To meet the $20 pCi/m^2$-sec standard, equation (9) from Appendix P must be used to determine the cover thickness:

$$x_1 = 2.28\exp(-013M_1)\left\{ \ln\left(\frac{2J_o}{J_1}\right) - \ln\left[\left(1 + \frac{P_o\exp(0.13(M_1-M_o))}{P_1}\right) + \left(1 - \frac{P_o\exp(0.13(M_1-M_o))}{P_1}\right)\left(\frac{J_1}{J_o}\right)^2\right]\right\} \quad (1)$$

where

x_1 = required depth of cover in meters

M_o = weight-percentage of moisture in tailings

M_1 = weight-percentage of moisture in soil cover

J_o = radon flux at the surface of the bare tailings (pCi/m^2-sec)

J_1 = radon flux from the surface after attenuation with the cover (pCi/m^2-sec)

P_o = porosity of the tailings (dimensionless)

and P_1 = porosity of the cover material

The radon flux from the bare tailings, J_o, is calculated from equation (14) from Appendix P:

$$J_o = [Ra]\rho E(\lambda D_o/P_o)^{1/2} \times 10^4 \quad (2)$$

where

$[Ra]$ = concentration of radium-226 in the tailings solids (pCi/g)

ρ = density of the tailings solids (g/cm^3)

E = emanating power of tailings (dimensionless)

D_o = effective bulk diffusion coefficient for radon in the tailings (cm^2/sec)

P_o = porosity or void fraction in tailings solids (dimensionless) and λ = decay constant of radon ($2.1\times10^{-6}sec^{-1}$).

On the assumption that no radium is removed with U_3O_8 from the ore in the mill process, [Ra] can be determined from the average ore grade. From the start of 1980 to 30 September 1985 the ore grade has averaged 0.2149% U_3O_8 and for 1985 only (nine months) it averaged 0.2036%. From a radiological standpoint, future grades are expected to be no worse; and, the radium of concern is that quantity which is nearer to the surface of the tailing piles. Therefore, 0.2149% U_3O_8 will be used as the ore grade for determining the radium concentration in the tailing solids:

$$[Ra] = K_aG$$
$$= 604.3 \text{ pCi/g}$$

where

G = ore grade (weight % U_3O_8)

K_a = 2812 pCi Ra-226 per gram tailings solids/(weight-% U_3O_8).

The in-place dry density of the tailings solids was measured at 1.652 g/cm^3 and, as indicated in Section 3.0 of Appendix P (GEIS), the emanating power of the tailings can be assumed at 0.2. The diffusion coefficient, D_o/P_o, can be found from equation (1) in Appendix P if the moisture content, M_o, is known. M_o was measured at 20.7%. Thus:

$$D_o/P_o = 0.106\exp(-0.261M_o) \quad (3)$$
$$= 0.000477 \text{ cm}^2\text{/sec}$$

Now, the radon flux from the bare tailings can be found by substituting the above values in equation (2):

$$J_o = [604.3](1.652)(0.2)((2.1\times10^{-6})(0.000477))^{1/2}\times10^4$$
$$= 63.22 \text{ pCi/m}^2\text{-sec}$$

With J_1 now fixed by regulation at 20 pCi/m^2-sec, setting $P_o = P_1$, and using 7.0% for the ultimate moisture content of the soil cover (M_1), the cover thickness required to reclaim RAMC's tailing piles can be found by substituting values in equation (1):

$$x_1 = 2.28\exp(-0.13(7.0))\left\{\ln\left(\frac{2(63.22)}{20}\right) - \ln\left[\left(1 + \exp(0.13(7.0-20.7))\right) + \left(1 - \exp(0.13(7.0-20.7))\right)\left(\frac{20}{63.22}\right)^2\right]\right\}$$

$$= 1.486 \text{ meters} = \underline{\underline{4.876 \text{ feet}}}.$$

In the same manner, cover thickness can be determined for other ore grades and tailing/cover moistures. Table 5.5-11 shows the effect of each. However, as with the tables in Appendix P, cover porosity is assumed equal to that of tailings ($P_o = P_1$) and no allowance is made for the effect of soil cover type or its compaction on the diffusion coefficient. An attempt shall be made, therefore, to allow for those parameters in the following calculations to see if they have any effect on the amount of cover required for RAMC's tailings piles. NUREG/CR-3533 shall be used for that purpose (Ref. 4).

Table 5 (Ref. 4) gives a procedural checklist for calculating adequate cover thickness. The first step is to determine the source parameters, R, E, ρ_t, g_t, p_t, a_t, and b_t where the subscript t refers to the tailings:

R = 604.3 pCi/g

E = 0.2

ρ_t = 1.652 g/cm^3, the in-place dry bulk density

g_t = 2.800 g/cm^3, the tailing-particle density

$$p_t = 1 - \frac{\rho_t}{g_t}$$

TABLE 5.5-11
REQUIRED DEPTHS FOR RADON ATTENUATION (FEET)

Mo Tailings Moisture (%)	Diffusion Coefficient (cm^2/sec)	Cover Moisture (%)/Diffusion Coefficient (cm^2/sec) M_1% / D/P	3.0 / .0484	5.0 / .0287	7.0 / .0171	9.0 / .0101	11.0 / .0060	13.0 / .0036	15.0 / .0021
				0.05% U_3O_8 Ore Grade					
5.0	.0287		9.38	6.77	4.82	3.36	2.29	1.52	0.96
9.0	.0101		7.58	5.54	3.99	2.82	1.95	1.30	0.83
13.0	.0036		5.13	3.79	2.77	1.99	1.40	0.96	0.63
17.0	.0013		1.54	1.16	0.86	0.63	0.46	0.32	0.22
21.0	.0004		0	0	0	0	0	0	0
25.0	.0002		0	0	0	0	0	0	0
				0.10% U_3O_8 Ore Grade					
5.0	.0287		12.90	9.48	6.89	4.96	3.52	2.46	1.69
9.0	.0101		11.22	8.31	6.10	4.43	3.17	2.24	1.54
13.0	.0036		9.14	6.83	5.06	3.71	2.69	1.91	1.34
17.0	.0013		6.53	4.92	3.68	2.73	2.00	1.45	1.03
21.0	.0004		3.05	2.32	1.75	1.32	0.98	0.72	0.52
25.0	.0002		0	0	0	0	0	0	0
				0.15% U_3O_8 Ore Grade					
5.0	.0287		14.96	11.06	8.11	5.90	4.24	3.02	2.11
9.0	.0101		13.29	9.90	7.32	5.37	3.89	2.79	1.97
13.0	.0036		11.29	8.47	6.32	4.67	3.42	2.47	1.76
17.0	.0013		8.91	6.74	5.06	3.77	2.79	2.04	1.47
21.0	.0004		6.00	4.56	3.45	2.60	1.94	1.44	1.05
25.0	.0002		2.15	1.64	1.25	0.95	0.72	0.54	0.40
				0.20% U_3O_8 Ore Grade					
5.0	.0287		16.42	12.18	8.98	6.56	4.76	3.41	2.42
9.0	.0101		14.76	11.03	8.19	6.04	4.41	3.19	2.28
13.0	.0036		12.78	9.62	7.20	5.35	3.94	2.87	2.07
17.0	.0013		10.49	7.94	5.98	4.48	3.33	2.45	1.78
21.0	.0004		7.82	5.95	4.51	3.40	2.55	1.89	1.39
25.0	.0002		4.47	3.42	2.61	1.98	1.50	1.12	0.84
				0.25% U_3O_8 Ore Grade					
5.0	.0287		17.55	13.06	9.65	7.08	5.15	3.72	2.66
9.0	.0101		15.89	11.91	8.87	6.55	4.81	3.49	2.51
13.0	.0036		13.93	10.50	7.87	5.87	4.34	3.18	2.31
17.0	.0013		11.68	8.85	6.68	5.02	3.74	2.76	2.02
21.0	.0004		9.12	6.94	5.27	3.98	2.99	2.23	1.65
25.0	.0002		6.06	4.63	3.53	2.68	2.03	1.53	1.14
				0.30% U_3O_8 Ore Grade					
5.0	.0287		18.47	13.77	10.20	7.50	5.48	3.97	2.85
9.0	.0101		16.82	12.62	9.41	6.98	5.13	3.74	2.71
13.0	.0036		14.86	11.22	8.43	6.29	4.67	3.43	2.50
17.0	.0013		12.63	9.59	7.25	5.45	4.07	3.02	2.22
21.0	.0004		10.14	7.73	5.87	4.44	3.34	2.50	1.85
25.0	.0002		7.25	5.55	4.23	3.22	2.44	1.84	1.37

= 0.410, the in-situ tailing dry porosity (dimensionless)

$$m_t = \frac{M_t}{100}\left(\frac{1}{\rho_t} - \frac{1}{g_t}\right)^{-1}$$

$$= \frac{20.7}{100}\left(\frac{1}{1.652} - \frac{1}{2.800}\right)^{-1}$$

= 0.834, moisture saturation, or fraction of tailing pore space filled with water (dimensionless)

$$D_t = 0.07\exp(-4(m_t - m_t p_t^2 + m_t^5))$$

= 0.000869 cm^2/sec, the diffusion coefficient for radon in the total pore space of bulk tailings (=D/P in the GEIS)

$$a_t = p_t^2 D_t (1 - 0.74m_t)^2$$

$$= 21.41\times10^{-6}\ cm^2/sec$$

$$b_t = (\lambda / D_t)^{1/2}$$

$$= (2.1\times10^{-6}/0.000869)^{1/2}$$

$$= 0.049159\ cm^{-1}$$

For an infinite tailing thickness, or depth, the bare tailing flux is determined from equation (2) which, re-stated and with values inserted, is as follows:

$$J_t = 10^4 . R \rho_t E(\lambda D_t)^{1/2}$$

$$= 10^4(604.3)(1.652)(0.2)((2.1\times10^{-6})(0.000869))^{1/2}$$

$$= 85.29\ pCi/m^2\text{-}sec$$

or about 35% higher than already determined for the same quantity, J_o. It can be seen that the difference lies solely with the change in radon diffusion coefficient.

However, RAMC has tailings which vary in thickness from zero up to approximately 70 feet. The effect of tailings depth on bare tailings surface flux emanation can be accounted for in the formula:

$$J_t = 10^4 . R \rho_t E(\lambda D_t)^{1/2} \tanh \left(\frac{\lambda}{D_t} \right)^{1/2} x_t \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

$$= 85.29 \tanh 0.049159 x_t \text{ pCi/m}^2\text{-sec}$$

where

x_t = tailings thickness in cm.

If equation (4) is evaluated for different tailings thicknesses, the effect of tailings depth on flux emanation can be seen--as shown in Table 5.5-12:

TABLE 5.5-12
TAILINGS THICKNESS vs BARE TAILINGS FLUX

Thickness, x_t		Flux, J_t	Thickness, x_t		Flux, J_t
cm	ft.	pCi/m^2-sec	cm	ft.	pCi/m^2-sec
3.05	0.1	12.69	60.96	2	84.87
6.10	0.2	24.82	91.44	3	85.27
9.14	0.3	35.95	121.92	4	85.29
12.19	0.4	45.77	152.40	5	85.29
15.24	0.5	54.13	304.80	10	85.29
30.48	1.0	77.18	2133.60	70	85.29

The proposed earthen cover shall consist of a six-inch thick layer of fine- to medium-grained sand overlain by on-site soil. The purpose of the sand layer will be to act as a capillary break between the tailings and soil cover. This capillary break will prevent the flow of moisture and contaminants from the tailings to the soil cover, thus precluding long-term contamination of the cover. Sand to be used for the capillary break will be obtained from the Keystone-Wallace copper tailings pile located approximately two miles southeast of the Lisbon plant site. Figure 5.5-1 shows the gradation range for this sand, which may be compared against that of RAMC's slimes tailings. The figure also shows the average gradation for typical on-site earth fill which is proposed as the main cover material. The on-site soil is a reddish-brown silty clay and clayey silt which is derived from the residual slope wash and alluvium soils in the site vicinity.

No practicable method is known to effectively dewater and/or consolidate the present tailing piles. In addition to the difficulties of operating heavy equipment on the tailing piles, this fact forces RAMC to use an unorthodox method of emplacing the cover material. RAMC proposes to reclaim the piles by slurrying the cover material with clean water and placing it over the tailings in the same way that the tailing is deposited.

The sand will be hauled by self-loading scrapers or road-trucks to a convenient nearby location. A slurrying system comprising of a feeder, agitator and tank, and slurry pump and associated piping will be used to place the sand blanket. The sand blanket cover slope will be controlled by controlling the slurry density and discharge points in the same manner that is used during operation for forming tailing beaches. Slopes can be adjusted this way to account for such factors as particle size separation and associated radon attenuation factor changes.

Following completion of the sand blanket capillary break, the proposed slurrying system will be moved to the on-site borrow area shown in Figure 5.5-2. However, in order to ensure that no disturbance of cultural resources occurs, an archaeological and historical artifact survey of the borrow area shall be performed prior to disturbance of the area. These surveys will be submitted to the USNRC and no such disturbance shall occur until the licensee has received authorization from the USNRC to proceed. In addition, all work in the immediate vicinity of any buried cultural deposits unearthed during the disturbance of land shall cease until approval to proceed has been granted by the USNRC. After clearing the borrow area of plant growth and top soil, self-loading scrapers and a front-end loader will be used to haul the fill to the slurrying system. The soil slurry will then be pumped, up to a maximum distance of 4,000 feet, to where it is needed. The same methods of control used for placing tailings and as described above shall be employed. Although there are convenient on-site deposits of such impervious materials as Mancos Shale and the slurrying method permits the easy emplacement of thin layers of such materials, no allowance will be made for them in this reclamation plan. (Clay layers could effectively increase soil cover long-term equilibrium moisture, thus greatly reducing the overall cover diffusion coefficient and cover thickness needed.)

In-situ densities of typical on-site soils were taken when it was necessary to determine the collapse potential of the natural earth at the toe of the lower tailings embankment (Ref. 5, Table 2). Those measurements indicated that undisturbed dry densities of naturally compacted earth could average as high as 98.38 lb/ft^3 (1.577 g/cm^3), or 81% of maximum dry density. It may, therefore, be reasonable to assume that 70% of maximum dry density could be achieved shortly after (say, two years) emplacement by the slurrying method; especially when the material is deposited at higher moistures than optimum. Therefore, for this particular reclamation plan, 70% and 80% of maximum dry density shall be used in calculations for "initial" (after about two years) and "ultimate" (after about 1,000 years) values, respectively, for both the capillary break sand and the cover soil. However, before proceeding with the implementation of the actual reclamation plan, RAMC plans to conduct additional in-place density and moisture tests of the actual borrow material to be used.

The second step (in Table 5, Ref. 4) is to determine the cover material parameters. The same terminology shall be used as in the first step with the subscript s representing the sand layer and the subscript c the main soil cover. The required soil thickness shall be calculated for two conditions: the "initial", and the "ultimate".

Since the long-term equilibrium moisture content is so important to the long-term (ultimate) thickness of cover required, additional in-situ testing of borrow soil must be conducted rather than relying upon one "crude" test for moisture (7.0%). Therefore, instead of using that moisture content, equilibrium moisture shall be estimated from the empirical relationship (equation (16), Ref. 4):

$$M = 3.1P^{1/2} - 0.03E + 3.9f_{cm} - 1.0 \qquad (5)$$

$$= 3.1(12.4)^{1/2} - 0.03(40.3) + 3.9(0.65) - 1.0$$

$$= 11.24\%$$

where

M = the dry weight percent soil moisture

P = annual precipitation (in.) - See Table 2.2-1

E = annual lake evaporation (in.) - See Table 4.2-3

f_{cm} = soil fraction passing a No. 200 mesh. - See Table 5.5-1.

From the foregoing, all cover material parameters are summarized as shown in Table 5.5-13.

For $x_t > 120$cm ($\sim$4 ft.) $\tanh b_t x_t \rightarrow 1$ and simplifies equation (4) of Reference 4 to:

$$J_s = \frac{2J_t \exp(-b_s x_s)}{\left[1+(a_t/a_s)^{1/2}\right] + \left[1-(a_t/a_s)^{1/2}\right] \exp(-2b_s x_s)} \qquad (6)$$

TABLE 5.5-13
COVER MATERIAL PARAMETERS

	Sand Layer		Soil Cover		
	Initial	Ultimate	Initial	Ultimate	Units
Particle density (estimated), g	2.550	2.550	2.550	2.550	g/cm^3
Maximum dry density (measured), ρ max	1.948	1.948	1.948	1.948	g/cm^3
Packing density (estimated), ρ	1.364	1.558	1.364	1.558	g/cm^3
In-situ dry porosity, $p=(1-\rho/g)$	0.236	0.389	0.236	0.389	Dimensionless
Dry wt.-% moisture, M	12.00 (est.)	8.57	15.00 (est.)	11.24	%
Fraction of saturation, $m=\frac{M}{100}\left(\frac{1}{\rho}-\frac{1}{g}\right)^{-1}$	0.352	0.343	0.440	0.451	Dimensionless
Diffusion coefficient, $D=0.07\exp(-4(m-mp^2+m^5))$	0.018125	0.021437	0.012435	0.014082	cm^2/sec
$a=p^2D(1-0.74m)^2$	0.000552	0.001806	0.000315	0.000946	cm^2/sec
$b=(\lambda/D)^{1/2}$	0.010764	0.009898	0.012995	0.012212	cm^{-1}

= 81.54pCi/m^2-sec for the initial condition,

and = 82.98pCi/m^2-sec for the ultimate condition, giving the radon fluxes expected from the sand blanket surface.

In order to calculate the thicknesses of cover required, it is first necessary to calculate an equivalent source diffusion coefficient, D_{t1}, from equation (7):

$$D_{t1} = D_t \exp(-b_s x_s) + D_s(1 - \exp(-b_s x_s)) \quad \ldots\ldots (7)$$

$$= (0.000869)(0.8487) + (0.018125)(1 - 0.8487)$$

$$= 0.003480 \text{ cm}^2/\text{sec for the initial condition}$$

and $$D_{t1} = (0.000869)(0.8600) + (0.021437)(1 - 0.8600)$$

$$= 0.003749 \text{ cm}^2/\text{sec for the ultimate condition.}$$

D_{t1} is now used to analyze the second layer (main soil cover). Since the cover consists of only two layers, J_s may be used as J_t and D_{t1} as D_t to calculate the required thickness of this second layer (Ref. 4, page 2-14). The new source term thickness is $x_s + x_t$ and D_{t1} is used to compute new values for a_t and b_t, say a_{t1} and b_{t1}:

$$a_{t1} = p_t^2 D_{t1}(1 - 0.74m_t)^2$$

$$= 85.74\text{x}10^{-6} \text{ cm}^2/\text{sec for the initial condition}$$

and $$a_{t1} = 92.37\text{x}10^{-6} \text{ cm}^2/\text{sec for the ultimate condition}$$

$$b_{t1} = (\lambda / D_{t1})^{1/2}$$

$$= 0.024565 \text{ cm}^{-1} \text{ for the initial condition}$$

and $$b_{t1} = 0.023667 \text{ cm}^{-1} \text{ for the ultimate condition.}$$

The required cover thickness for the two conditions can now be computed by using the similarly simplified ($x_t > 120$, $\tanh a_t x_t \rightarrow 1$) equation (8) of Reference 4:

$$x_c = \frac{1}{b_c} \ln\left[\frac{2J_s/J_c}{\left[1 + (a_{t1}/a_c)^{1/2}\right] + \left[1 - (a_{t1}/a_c)^{1/2}\right](J_c/J_s)^2}\right] \quad \ldots\ldots (8)$$

$$= 127.7 \text{ cm} = 4.19 \text{ ft. for the initial condition}$$

and $$x_c = 149.2 \text{ cm} = 4.90 \text{ ft. for the ultimate condition.}$$

The differences in cover material required by each condition may be better related by the weight of material required per unit area of tailings surface:

that is, $4.19 \text{ ft.} \times 121 \text{ lb/ft}^3 \times 0.70 = 355 \text{ lb/ft}^2 = 1.73 \text{ MT/m}^2$

and $4.90 \text{ ft.} \times 121 \text{ lb/ft}^3 \times 0.80 = 474 \text{ lb/ft}^2 = 2.31 \text{ MT/m}^2$

However, if just one parameter is changed, the resulting thicknesses required are drastically affected. For example, assume that the estimated specific gravity of the tailings, g_t, is changed from 2.800 to 2.600 and only the parameters that depend upon g_t are altered. The tailings moisture saturation, m_t, would be increased to 0.938 (may be closer to actual), the tailings diffusion coefficient, D_t, would drop to 0.000148 cm^2/sec and the source flux would drop from 85.29 to 35.20 pCi/m^2-sec. The resulting thicknesses of soil cover required to meet the 20 pCi/m^2-sec standard would then be 2.00 ft. and 2.40 ft. for the initial and ultimate conditions, respectively. Thus illustrating that a more thorough study of tailings and cover material parameters is needed to make effective use of NUREG/CR-3533 (Ref. 4)--otherwise the exercise is only academic. Therefore, until more reliable information is available, 5 ft. shall be used for the soil cover required in this reclamation plan.

The attenuation of external radiation (mainly gamma-rays) can be determined using various sources. EPA (1976) and Bear Creek ES (NRC, 1977) quote that one-foot of 1.6 g/cm^3 density soil cover will reduce external radiation by 90%. Using the estimated electron density of the 1.6 g/cm^3 (100 lb/ft^3) soil cover, it can be calculated that one-foot thickness of this soil will reduce external radiation by about 95%. And by assuming the most conservative case, which is a reduction factor of 90% per one-foot of 1.6 g/cm^3 soil cover, it can be shown that a 2.3-foot cover over the tailings will reduce the external radiation dose to about the background rate.

Erosion Analysis

An erosion analysis shall now be undertaken to show that the proposed cover materials will provide long-term isolation of tailings. The basic factors which directly affect sheet erosion are rainfall, soil properties, slope length, slope steepness, and type and condition of cover. The Universal Soil Loss Equation (USLE) is the standard equation which is used to predict the potential for and quantity of erosion from a given area.

The complete USLE equation is:

$$A = RKLSC \qquad (9)$$

where

A = the computed annual soil loss (sheet and rill) in tons per acre. A is not the sediment yield.

R = the rainfall factor: the number of erosion index units in a normal year's rain.

K = the soil erodibility factor: the erosion rate per erosion index unit for a specific soil in a cultivated continuous fallow on a 9% slope 72.6 feet long. The resistance of a soil surface to erosion is a function of the soil's physical and chemical properties.

L = the slope length factor: the ratio of the soil loss from the field slope length to that from a 72.6-foot length on the same soil type and gradient.

S = the slope gradient factor: the ratio of the soil loss from the field gradient to that from a 9% slope on the soil type and slope length.

C = the management factor: the ratio of soil loss with specified crop or vegetative cover to that of fallow condition from which the K factor is evaluated.

The R factor for the La Sal area is less than 20 (Wischmeier, 1976). However, to be conservative, a value of 20 will be utilized.

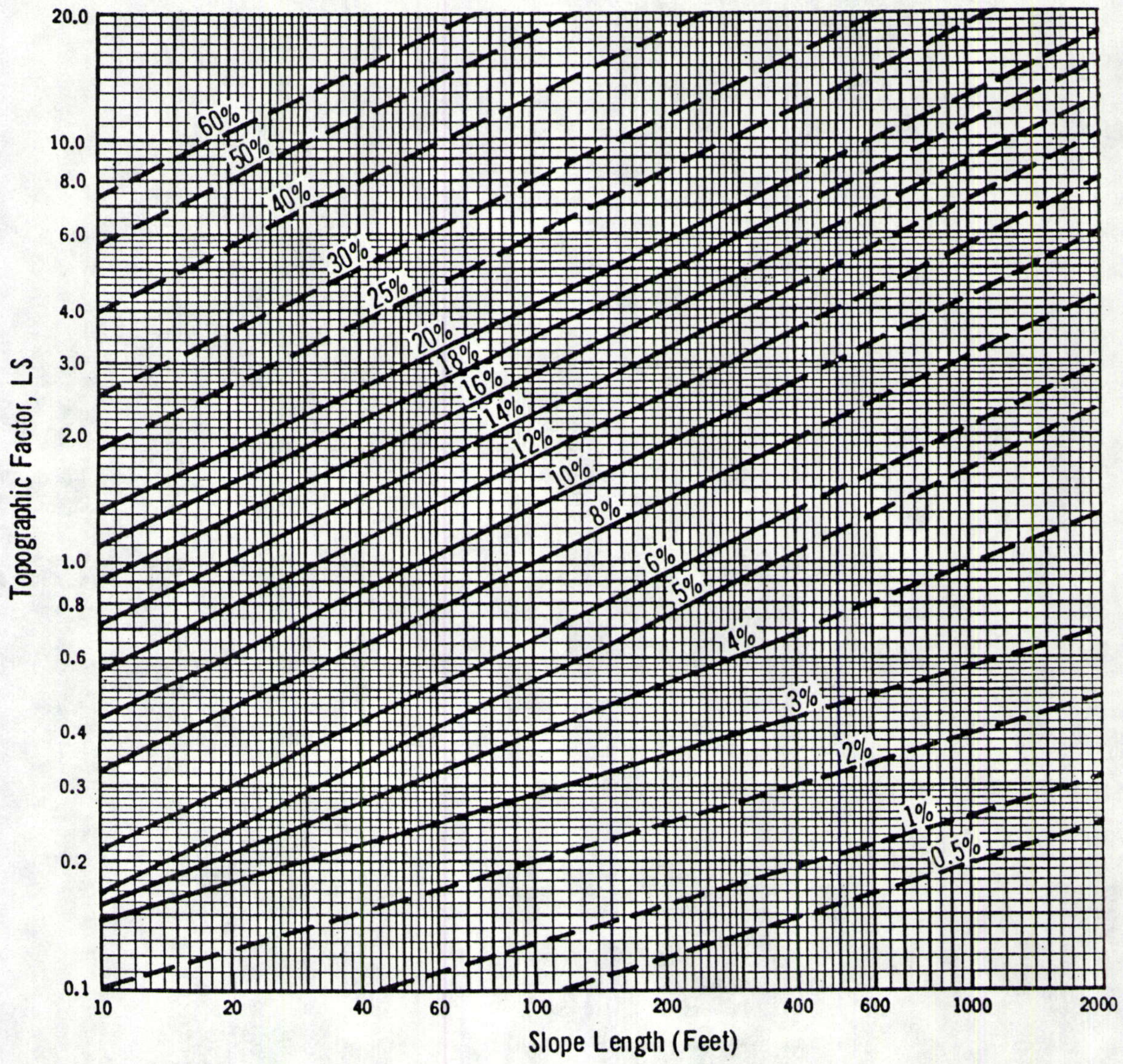
The soil erodibility factor for a sandy, silt loam can range from a low of 0.20 to an extreme of 0.49. The erodibility factor is totally dependent on the physical and chemical properties of the soil. An average value of .43 will be used to represent the K factor.

The combined slope length and steepness factor is 0.402. This factor is available from nomographs developed by the USDA, Soil Conservation Service (Figure 5.5-3). This factor is based on 1,000-foot slope lengths and 2% slope steepness.

The C factor was determined for permanent range and idle land with tall grass, weeds and short brush having an average drop height of 20 inches and 75% permanent cover. The grass cover which contacts the ground surface would be approximately 40%. For these conditions, the C factor is 0.06.

Using the above outlined factors and the USLE, the average annual soil loss due to sheet and rill erosion is projected to be 0.207 tons per acre per year. Even though this amount may erode, the sediment yield from the site would be significantly less (i.e., some of the eroded sediment will be redeposited on site). However, to be conservative, it is assumed that the actual sediment yield is equal to projected soil loss. Over a 1,000-year period, this would amount to approximately 207 tons being lost from a one-acre area.

Assuming an in-place density of 96 pounds per cubic foot, the average soil loss is 4,310 cubic feet per acre over a 1,000-year period. This is equal to an erosion rate of approximately 0.1 foot of soil from each acre over the 1,000-year design life of the cover. Thus, it is not anticipated



*The dashed lines represent estimates for slope dimensions beyond the range of lengths and steepnesses for which data are available. The curves were derived by the formula:

$$LS = \left(\frac{\lambda}{72.6}\right)^m \left(\frac{430x^2 + 30x + 0.43}{6.57415}\right)$$

where λ = field slope length in feet and m = 0.5 if s = 5% or greater, 0.4 if s = 4%, and 0.3 if s = 3% or less; and x = sin θ. θ is the angle of slope in degrees.

FIGURE 5.5-3 SLOPE-EFFECT CHART

that sheet and rill erosion will negatively affect the long-term stabilization of the reclamation project.

Gully erosion of the reclaimed cover will be prevented by placing all permanent drainages in riprap lined channels. Because of the gentle broad slopes associated with the tailings piles, the precipitation which does fall on the piles themselves will either infiltrate into the soil or flow as sheet flow across the reclaimed surface.

Wind erosion is not considered to be a significant problem associated with the long-term stabilization of the site. The site sets on a low, protected drainage and is not readily accessible by the prevailing winds. This condition, plus the use of a vegetative cover for final reclamation of the piles, suggests that wind erosion should not pose a significant problem to long-term stability of the site.

Flood Protection

The flood protection plan will be similar to that outlined in Section 4.2.1 and as illustrated in Figure 4.2-9 after abandonment. However, further investigations into the design of runoff-control facilities for this reclamation project indicate that two changes are necessary: the probable maximum precipitation (PMP) needs to be re-determined and a more realistic rainfall distribution for the western United States needs to be found.

Determination of PMP: A systematic methodology developed by Hansen et al. (1977) for the Colorado River and Great Basin drainages was used to derive estimates for the local-storm probable-maximum-precipitation event. By definition, probable maximum precipitation (PMP) is "the theoretically greatest depth of precipitation for a given duration that is physically possible over a particular drainage basin at a particular time of year" (American Meterological Society, 1959). In the southwestern United States, these events occur as intense thunderstorms over localized areas, thus requiring the addition of the terminology "local-storm" as opposed to the "general-storm" PMP that typically occurs over large areas in the eastern United States (Hansen et al., 1977--Reference 6).

From Figure 4.5 of Hansen et al. (1977), the local-storm one-hour PMP for a one square mile area at the Rio Algom site is 8.3 inches, which should be reduced by 5% for every 1,000 feet in elevation over 5,000 feet. With a site elevation of approximately 6,750 feet, this results in an elevation adjustment factor of 0.91.

From Figure 4.7 of Hansen et al. (1977), the duration adjustment factor for the site to convert from a one-hour to a six-hour PMP is 1.20. Hence, the six-hour local-storm PMP for Rio Algom is 9.1 inches [(8.3)(0.91)(1.20)].

As a comparison, the U. S. Bureau of Reclamation (1977) provides a map (reproduced here as Figure 5.5-4) that shows the one-hour local-storm PMP to be about 7.1 inches. This value compares reasonably well with the elevation-corrected one-hour local-storm PMP estimate of 7.6 inches [(8.3)(0.91)] from Hansen et al. (1977).

Riedel and Schreiner (1980) developed a map (see Figure 5.5-5) comparing the 10-square mile 6-hour PMP with the 100-year, 6-hour event. The ratio of the PMP to the 100-year storm from this figure is approximately 4.2. According to Miller et al. (1973), the magnitude of the 100-year, 6-hour storm at the site is 1.90 inches. With an adjustment factor of 0.88 to reduce the storm area from 10 square miles to 1 square mile (see Figure 4.9 of Hansen et

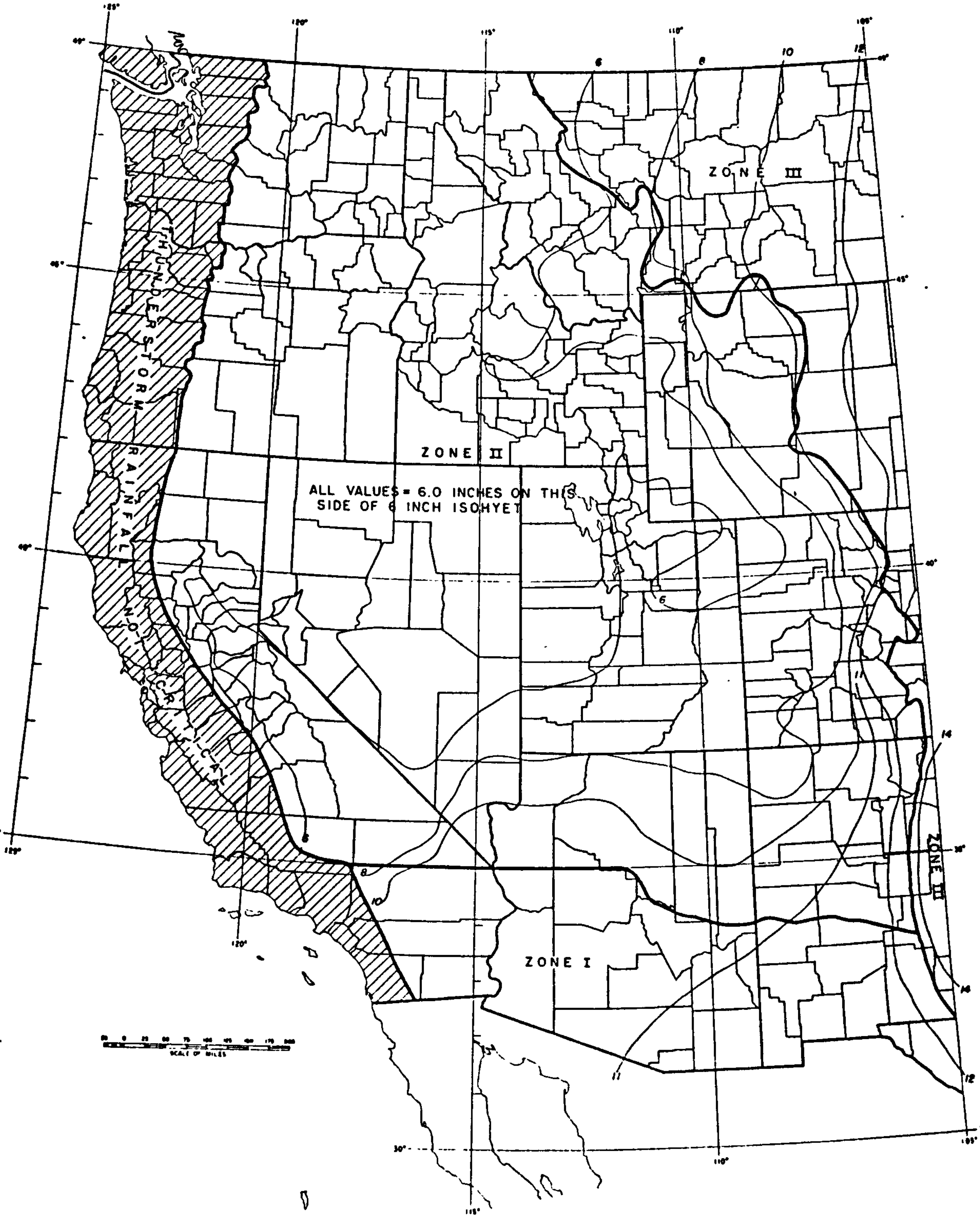


FIGURE 5.5-4 ONE-HOUR LOCAL-STORM PMP

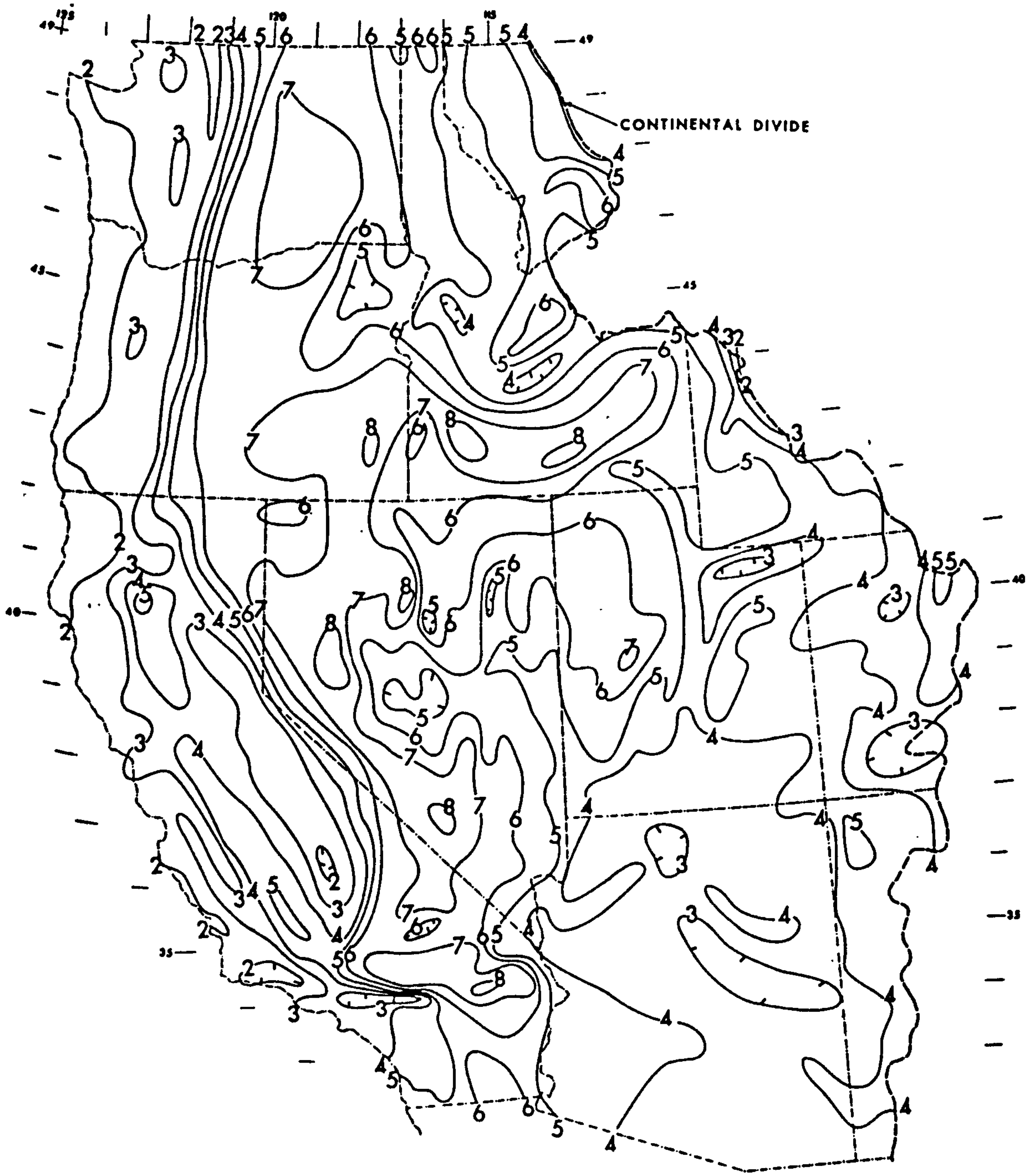


FIGURE 5.5-5 RATIO OF 10 SQUARE MILE 6-HOUR PMP TO 6-HOUR 100-YEAR STORM

al., 1977), the value of the 6-hour local-storm PMP from Riedel and Schreiner (1980) is 9.1 inches [(1.9)(4.2)/0.88]. This is equal to the value determined from Hansen et al. (1977).

Rainfall Distribution: Because of the influence of the rainfall hyetograph on the runoff hydrograph, an in-depth survey of rainfall mass curves was conducted to identify the curve that would be most indicative of conditions at the Rio Algom site. Several investigators have indicated that, in southeastern Utah and much of the remainder of the southwestern United States, large thunderstorms (the PMP being the ultimate example) are typified by a large percentage of the total rainfall occurring at the beginning of the storm during a small portion of the total storm duration.

Frederick et al. (1981), in a study of precipitation in the western United States, indicated that the major portion of a 6-hour precipitation event in the southwest begins in the first hour of the event. Hansen et al. (1977) similarly concluded that "a large portion of the total storm should occur in the first hour and almost all within 3 hours" of the beginning of a 6-hour PMP in the area occupied by the Rio Algom site.

The U.S. Weather Bureau (1947) examined thunderstorms throughout the United States and concluded that, for point rainfall (e.g., the local-storm PMP), "the time distribution of thunderstorm rainfall at a station is such that the highest intensity occurs at the beginning, with decreasing intensity throughout the storm." Nonetheless, the U.S. Weather Bureau (1947) and the U.S. Army Corps of Engineers (1952) derived mass curves for general storms that are so arranged to result in most of the rainfall occurring in the middle of the storm. Although the U.S. Army Corps of Engineers' (1952) curve has been suggested by the U.S. Nuclear Regulatory

Commission (1982) for use in analyzing runoff from the PMP, the curves were developed from data that are not typical of the Rio Algom site. The U.S. Army Corps of Engineers (1952) cautions that their publication is valid only for the eastern United States (east of the 105th meridian) while the mass curve derived by the U.S. Weather Bureau (1947) is based on data obtained from eastern Ohio where meteorologic conditions are significantly different than those found in southeastern Utah. Hence, neither of these curves is appropriate for use at the Rio Algom site.

Little information is available on actual storm distribution in the western United States. Keppel (1963) studied a single high-intensity event in eastern New Mexico and found that 89% of the total storm rainfall occurred within the first 25% of the storm duration. Renard (1970) and Renard and Simanton (1975) presented depth-duration data for three individual storms in eastern New Mexico and southeastern Arizona that showed similar mass-curve patterns (greatest depths at the onset of the storm).

Farmer and Fletcher (1972) examined mass curves from several storms in central, eastern, and southern Utah and established the concensus dimensionless storm distribution shown in Table 5.5-14. As is typical of high-intensity storms in the region, this dimensionless mass curve yields a majority of the total precipitation within the initial stages of a storm. Combined with the fact that the data used to develop this mass curve were collected in the general region of the Rio Algom site, this curve was used to model the time-distribution of the PMP in generating runoff hydrographs for the site.

The Farmer-Fletcher storm distribution (mass curve) is compared graphically against the "Type B" storm pattern of the U. S. Soil Conservation

TABLE 5.5-14 - FARMER-FLETCHER STORM DISTRIBUTION

Percent Time	Percent Rainfall	Interval Percent
0	0	
		36.5
10	36.5	
		25.0
20	61.5	
		15.4
30	76.9	
		7.0
40	83.9	
		4.1
50	88.0	
		2.8
60	90.8	
		2.4
70	93.2	
		2.0
80	95.2	
		2.5
90	97.7	
		2.3
100	100.0	

Service (1973), the mass curve used by RAMC in its license renewal application (December, 1982), and the distribution later recommended for licensees' use by the U. S. NRC (Staff Technical Position WM-8201, 1983) in Figure 5.5-6. These storm distributions were then used to compare the resulting peak runoff flow rates from subwatersheds above the tailings impoundments. Use of "SCS" and "NRC" curves result in average peak runoff rates 29% above and 28% below, respectively, compared with those obtained (Table 5.5-15) using the Farmer-Fletcher storm distribution. The Dames & Moore curve resulted in an average peak flow 170% above the Farmer-Fletcher.

Hydrograph Synthesis: With the design rainstorm (6-hour local-storm PMP hyetograph) established, it is now necessary to develop a runoff hydrograph for the total watershed above the tailings impoundment so that diversion

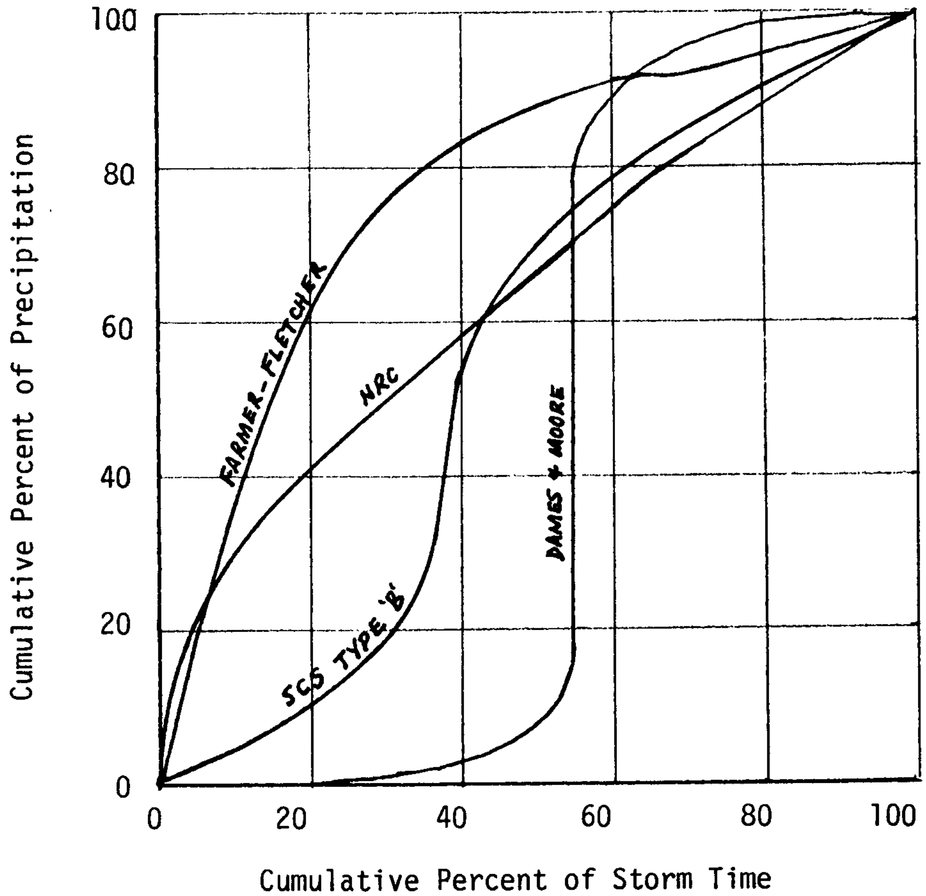


FIGURE 5.5-6 - COMPARISON OF PMP MASS CURVES

works can be correctly designed to handle the worst-case of storm-water runoff. (A runoff hydrograph is simply a plot of flow rate against time and is the result of a particular effective rainfall hyetograph, or rainfall excess, as modified by watershed flow characteristics.) To make the best possible hydrologic estimate, it is first necessary to develop runoff hydrographs for each (sub)watershed above the impoundments; this permits the individual watershed characteristics to be accounted for and allows for the determination of flow rates within the total watershed. Watershed boundaries and other pertinent conditions at the site are shown in Figure 5.5-2.

In summary, data obtained from those watersheds were input to a computer code developed by Hawkins and Marshall (1979, for the Utah Division of Oil, Gas and Mining) to generate runoff hydrographs from the PMP. The computer code (termed SCSHYDRO) models runoff using the rainfall-runoff function and triangular unit hydrograph of the U. S. Soil Conservation Service (1972). Results of computations are summarized in Table 5.5-15 for the 6-hour local-storm PMP event (9.1 inches) using the Farmer-Fletcher storm distribution.

TABLE 5.5-15 - RESULTS OF RUNOFF CALCULATIONS

	INPUT			OUTPUT			
Watershed	Area (square miles)	Curve Number (CN)	Time of Concentration (hours)	Runoff Depth (inches)	Initial Abstraction (inches)	Peak Flow (CFS)	Time to Peak (hours)
A-1	0.074	90	0.24	7.8903	0.2222	233.50	0.63
A-2	0.023	88	0.21	7.6461	0.2727	69.34	0.63
A-3	0.146	91	0.54	8.0121	0.1978	406.61	0.90
A-4	0.051	92	0.47	8.1337	0.1739	151.50	0.81
B-1	0.145	82	0.86	6.9097	0.4390	303.00	1.43
B-2	0.145	84	0.58	7.1558	0.3810	339.37	1.09
C-1	0.021	92	0.20	8.1337	0.1739	70.48	0.61
C-2	0.091	92	0.28	8.1337	0.1739	293.86	0.64
D-1	0.202	82	0.77	6.9097	0.4390	434.68	1.33
D-2	0.070	86	0.31	7.4013	0.3256	193.58	0.70

Excess rainfall (also known as runoff depth or effective rainfall) is defined as precipitation minus abstractions. Abstractions from precipitation are losses from precipitation that do not show up as storm water runoff. They include interception, evaporation, infiltration, surface storage, surface detention, and bank storage. Only initial abstractions (interception and surface storage) and infiltration are considered in this storm water management plan.

The Soil Conservation Service (SCS) of the U. S. Department of Agriculture (1971) combines infiltration losses with surface storage and estimates rainfall excess or equivalently the runoff volume (in inches) by the relationship

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (10)$$

which is valid only for rainfalls in excess of the initial abstraction ($P > 0.2S$). Q is the accumulated runoff volume (runoff depth in inches--for example, Table 5.5-15, column 5) for a storm of P inches (in this case, the PMP) and S is a measure of the watershed's storage capabilities represented by

$$S = \frac{1000}{CN} - 10 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (11)$$

where CN is known as a curve number. The values of 1000 and 10 carry the dimensions of inches and CN is only a convenient transformation of S (also in inches) to establish a scale of 0 to 100 and has no intrinsic meaning. Tables and charts of CN as a function of soil type, vegetative type, cover density, and moisture conditions are found in numerous publications.

Average curve numbers for the watersheds of concern were obtained by determining the area within each watershed occupied by various vegetative types (pinon-juniper and sagebrush-grass) and by surface disturbances such as the reclaimed site shortly after completion. This determination was based on maps (scale 1:4800) and aerial photographs of the site. Curve numbers for the individual vegetative types were obtained from Figure 5.5-7 using estimated cover densities of 20 percent for the pinon-juniper type and 50 percent for the sagebrush-grass type for antecedent-moisture-condition II (AMC II). From Figure 5.5-7, curve numbers for the pinon-juniper and sagebrush-grass

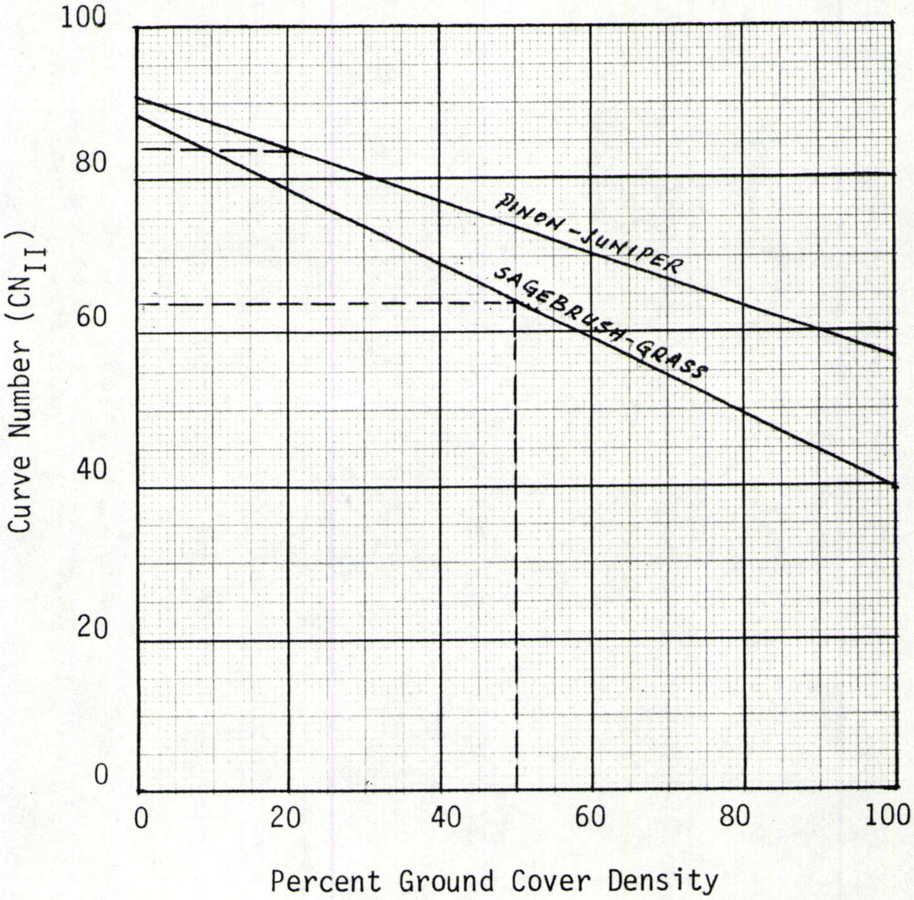


FIGURE 5.5-7 FOREST-RANGE CURVE NUMBERS FOR AMC II, SOIL GROUP C IN THE WESTERN UNITED STATES

vegetative communities were determined to be 84 and 64, respectively, assuming a hydrologic soil group of C (as is typical of much of the region, and as defined by SCS: "Soils having slow infiltration rates when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine texture. These soils have a slow rate of water transmission").

For the disturbed areas, curve numbers were chosen from tabulated values presented by the U. S. Soil Conservation Service (1972) using conservative judgement. Accordingly, a value of 80 was used for the reclaimed site (and values of 90 and 87 were used for the plant site and unreclaimed tailings, respectively, for the operational flood protection plan), again, for antecedent-moisture-condition II.

For each particular watershed, average curve numbers were determined by weighting the individual curve numbers according to area. These weighted values were then converted to antecedent-moisture-condition III using tabulations provided by the U. S. Soil Conservation Service (1972) to account for high soil-moisture contents just prior to storm occurrence, as suggested by the U. S. NRC (1982). For example, to determine the curve number for watershed A-1 which has 17.6 acres of pinion-juniper (CN=84), 19.4 acres of reclaimed site (CN=80) and the remaining area in sagebrush-grass (CN=64), in hydrologic soil group C and for AMC II, the weighted CN is

$$CN_{II} = [(17.6)(84) + (19.4)(80) + (10.6)(64)]\ /47.6 = 78$$

and $CN_{III} = 90$ (see U.S. SCS, 1972)

Two factors indicate that the curve numbers used in this analysis are conservatively high. First, the occurrence of antecedent-moisture-condition III in southeastern Utah is considered a rare event. In actuality, condition I is the norm, with condition II being the design condition. Hawkins (1973), in fact, found that antecedent soil moisture has little effect on the actual curve number for semiarid watersheds.

Second, as indicated by Hawkins (1973, 1979), the curve number is not a constant of the watershed but, rather, a variable that is inversely proportional to storm depth. Such a conclusion is also suggested by the U. S. Bureau of Reclamation (1977) in their tabulation of curve numbers for thunderstorm events, where the value of the suggested curve number decreases with increasing design storm size. With the PMP representing the largest possible storm at a given area, the actual curve number during such a storm will, therefore, be as small as possible for the site conditions, rather than being larger than "normal" as occurs when increased to account for antecedent-moisture-condition III.

The incremental runoff for any time period t to $t+\Delta t$ was determined from

$$\Delta Q = Q(t+\Delta t) - Q(t) \qquad (12)$$

$$= \frac{[P(t+\Delta t) - 0.2S]^2}{P(t+\Delta t) + 0.8S} - \frac{[P(t) - 0.2S]^2}{P(t) + 0.8S}$$

where S is determined from equation (11) by using the curve number for AMC III and values for $P(t+\Delta t)$ and $P(t)$ are determined from the rainfall mass curve (see Table 5.5-14).

Now that the effective rainfall hyetograph has been determined for each watershed, the runoff hydrographs can now be developed by making use of the watershed flow characteristics. By definition, the volume of water under an effective rainfall hyetograph is equal to the volume of surface runoff; this fact was used in the computer code to verify the runoff volume correctness.

The translation of the effective rainfall hyetograph (runoff depth) to an outflow hydrograph is accomplished in the code (Hawkins and Marshall, 1979) using the triangular unit hydrograph of the U. S. Soil Conservation Service (1972). This unit hydrograph is shown in Figure 5.5-8 along with a typical curvilinear hydrograph. It is characterized by its time to peak (T_p), recession time (T_r), time of base (T_b), and the relationship between these parameters. Thus, from the geometry of a triangle, the incremental

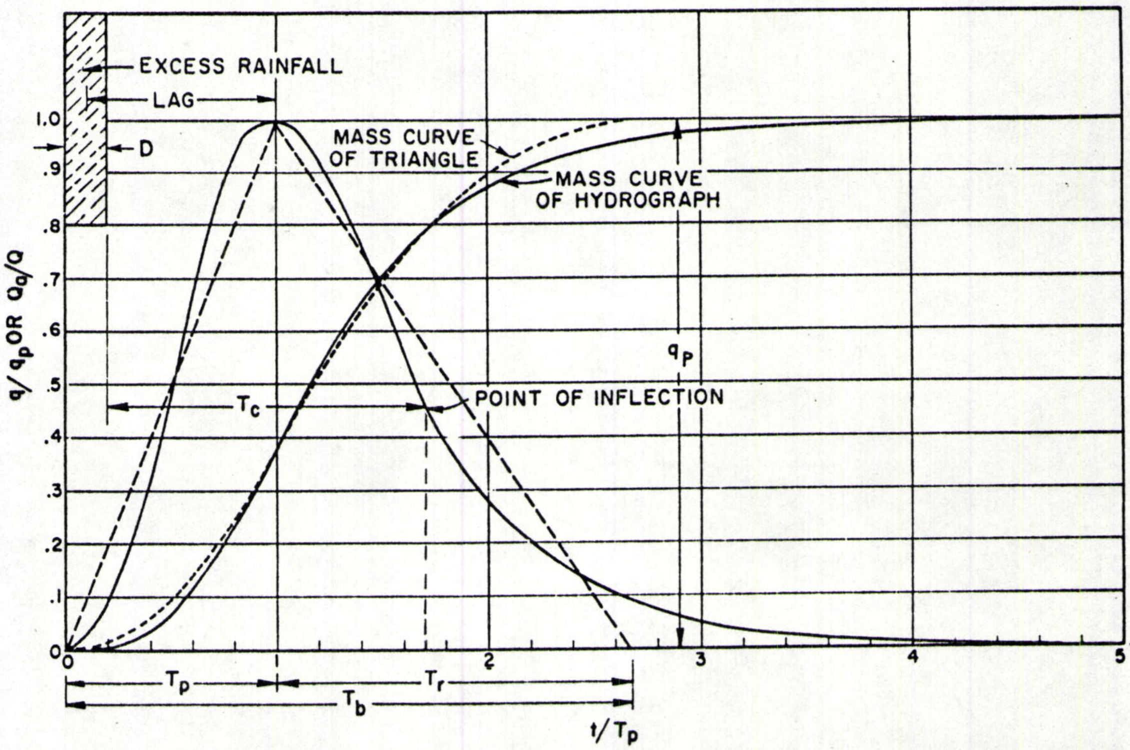


FIGURE 5.5-8 DIMENSIONLESS UNIT HYDROGRAPH AND MASS CURVE WITH TRIANGULAR UNIT HYDROGRAPH SUPERIMPOSED

runoff (ΔQ) can be defined by the equation

$$\Delta Q = \frac{(8/3)T_p \cdot q_p}{2} \quad \ldots \ldots \quad (13)$$

or

$$q_p = \frac{0.75\Delta Q}{T_p} \quad \ldots \ldots \quad (14)$$

where q_p is the peak flow rate (dimensioned according to Q and T) and other parameters have been previously defined: when Q is in inches and T_p in hours, q_p will be in inches per hour.

The flow rate at any time ($0 < t < T_b$) may be determined by simple linear proportioning of the triangular unit hydrograph. The time to peak is related to the familiar expression "time of concentration" (T_c) by the equation

$$T_c + \Delta t = 1.7\, T_p \quad \ldots\ldots \quad (15)$$

in which the factor 1.7 is an empirical finding cited by the U. S. Soil Conservation Service (1972).

The time of concentration may be estimated by any one of several formulas. For this flood analysis, T_c was determined from the following equations (SCS, 1972):

$$T_L = \frac{L^{0.8}(S + 1)^{0.7}}{1900\, Y^{0.5}} \quad \ldots\ldots \quad (16)$$

and

$$T_c = \frac{5}{3} T_L \quad \ldots\ldots \quad (17)$$

where T_L is the lag in hours, L is the hydraulic length of the watershed, or distance along the main channel to the watershed divide in feet, S is the watershed storage factor defined by equation (11), Y is the average watershed land slope in percent, and T_c is the time of concentration in hours.

The Soil Conservation Service (1972) shows that Δt must equal 0.2 T_p. Hence, from equation (15), the computer code used only T_c, and from this value computes T_p, T_r, Δt, and interim unit hydrograph ordinates. These unit hydrograph ordinates are calculated in the code under the array name H().

The mometary flow rate in inches per hour at any time t is then

$$q(t) = H(2)*q_p(t-1) + H(3)*q_p(t-2) + \ldots + H(14)*q_p(t-13)$$

$$= \sum_{i=1}^{14} H(i)*q_p(t-(i-1)) \quad \ldots\ldots \quad (18)$$

for all existing non-zero elements. Conversion to cubic feet per second is a simple calculation given by $q(cfs) = q(in/hr)*645.33*Area(mi^2)$.

Flow rate at any time during the PMP storm can be determined at selected stations by summing individual watershed hydrographs that contribute to the flow at that particular station (peak flows obtained from these composite hydrographs are considered to be conservatively high). For instance, hydrographs from watersheds B-1, B-2, D-1, and D-2 are combined as shown in Figure 5.5-9 to find the total flow entering the plant area at the present location of Bisco Lake embankment.

Flood diversion structures will be provided in the locations shown in Figure 5.5-10 to prevent erosion and thus ensure long-term isolation of tailings. Primary diversion will consist of a drainage basin/channel which runs from the present Bisco Lake embankment, through the present plant area, and to a spillway in the southwest corner of the drainage area; it will handle all runoff from above the upper tailings embankment. Two secondary diversion channels will be used within this drainage area: a riprap lined channel to divert concentrated runoff from the east side of the upper tailings impoundment, and contoured upper tailings cover to prevent over-embankment flow from watershed A-4. Concentrated flood flows below the upper embankment will be diverted away from the lower embankment by two channels: one on the southeast end, and one on the northwest end of the lower tailings embankment.

A drainage basin/spillway arrangement was chosen as the main diversion structure rather than a diversion channel as shown in Figure 4.2-9 for several reasons: it is the most economical method of controlling flood flows, including the PMF; it reduces the off-site impact of flood flows; it prevents the loss of any reclaimed-area sediments; and, because it is doubtful if sufficient on-site material will be available to construct such a channel after all contaminated material has been removed, borrow material would have

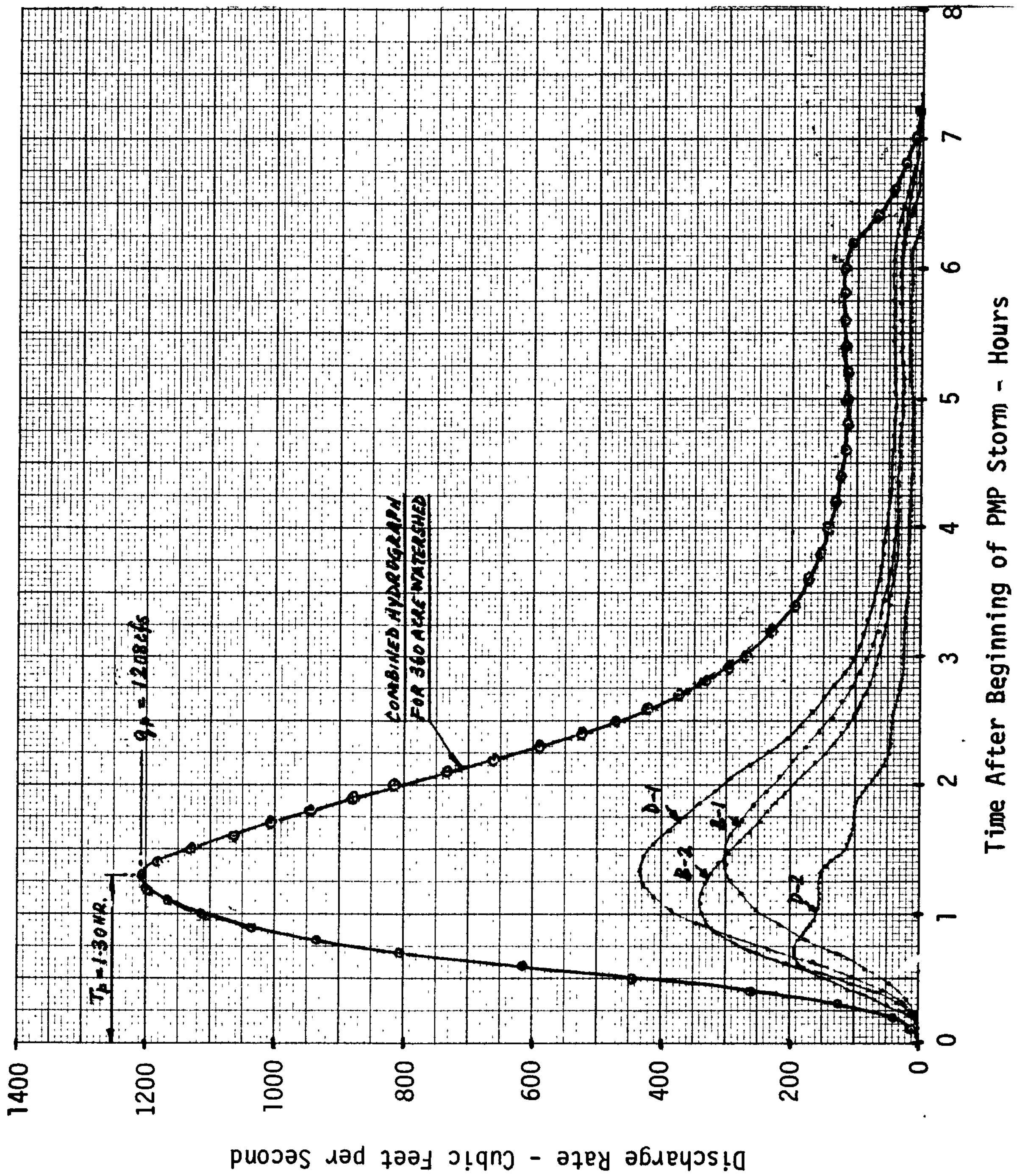


FIGURE 5.5-9 COMBINED HYDROGRAPH FOR 360 ACRE WATERSHED

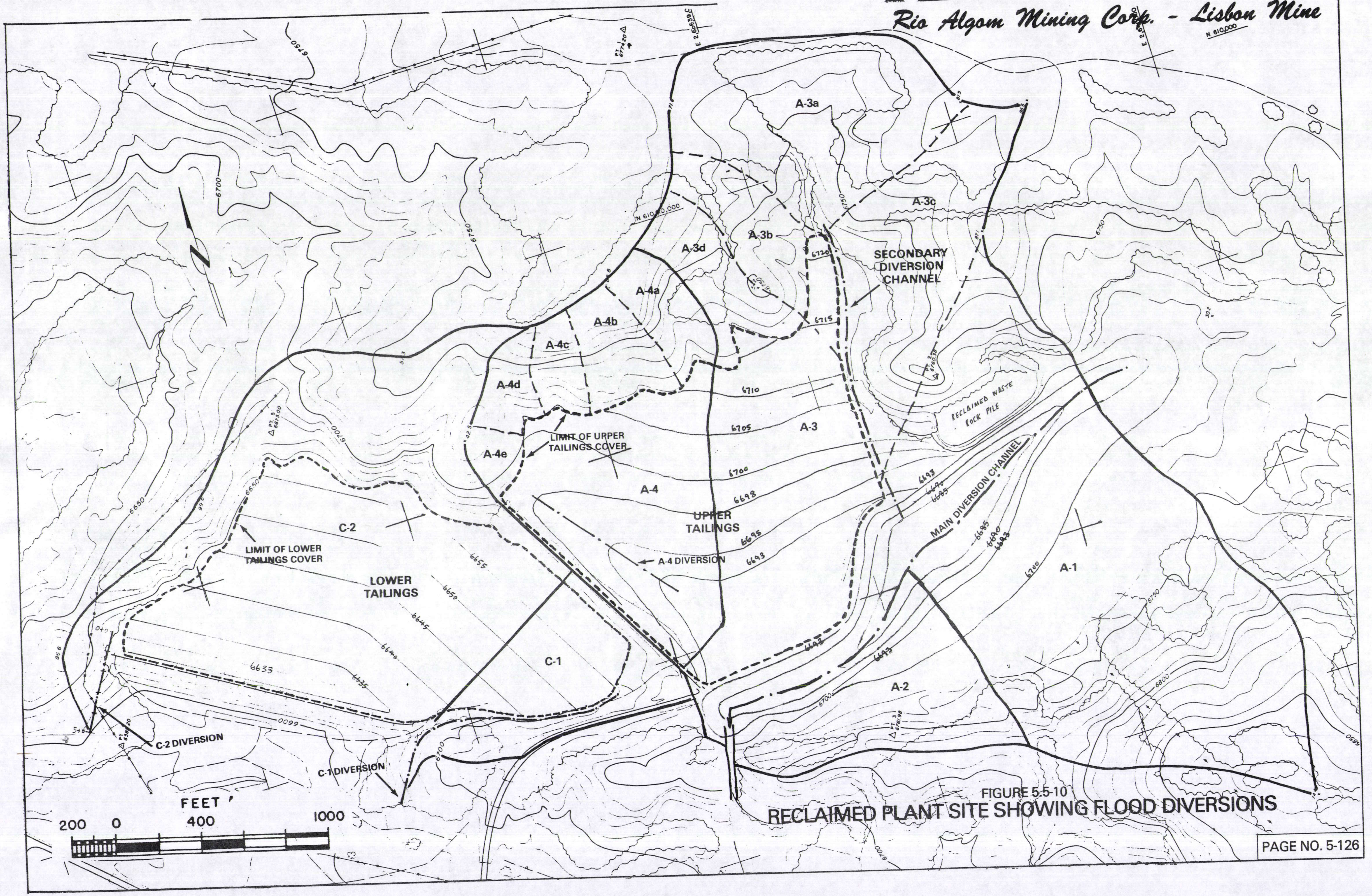


FIGURE 5.5-10
RECLAIMED PLANT SITE SHOWING FLOOD DIVERSIONS

to come from otherwise undisturbed land. The flood storage capacity of the proposed drainage basin is shown in Table 5.5-16. Hydrographs from watersheds A-1, A-2, A-3, and A-4 were combined with that from the 360-acre watershed (Figure 5.5-9), as shown in Table 5.5-17, to determine the spillway inflow hydrograph. A spillway must now be designed which will provide the desired discharge characteristics.

TABLE 5.5-16 - DRAINAGE BASIN STORAGE CHARACTERISTICS

ELEVATION feet	AREA acres	ΔV acre-feet	STORAGE acre-feet	STAGE feet	STAGE-STORAGE acre-feet
6685	6.38	0	0	0	0
6686	7.25	6.82	6.82	0	0
6687	8.19	7.72	14.54	0	0
6688	9.14	8.67	23.20	0	0
6689	10.17	9.66	32.86	0	0
6690	11.39	10.78	43.64	0	0
6691	12.80	12.10	55.73	0	0
6692	14.56	13.68	69.41	0	0
6693	{17.17	15.87	85.28	1	15.87
	{27.54				
6694	30.88	29.21	114.49	2	45.08
6695	33.73	32.31	146.79	3	77.38
6696	37.09	35.41	182.20	4	112.79
6697	40.36	38.73	220.93	5	151.52
6698	44.31	42.34	263.26	6	191.85
6699	47.82	46.06	309.33	7	239.92

TABLE 5.5-17 - SPILLWAY INFLOW HYDROGRAPH

TIME/minutes	360 ACRE/cfs	A-1/cfs	A-2/cfs	A-3/cfs	A-4/cfs	COMBINED/cfs
0	0	0	0	0	0	0
6	11	4	1	5	3	24
12	40	44	12	25	12	133
18	124	112	35	69	35	375
24	260	179	53	136	66	694
30	445	214	63	224	98	1044
36	617	232	69	295	127	1340
42	805	227	66	356	144	1598
48	937	199	58	394	151	1739
54	1037	180	53	405	147	1822
60	1118	173	52	398	139	1880
66	1165	174	53	384	132	1908
72	1200	175	53	368	127	1923
78	1208	163	48	351	120	1890
84	1182	136	39	329	113	1799
90	1129	116	34	305	103	1687
96	1064	110	33	278	94	1579
102	1004	110	33	258	86	1491
108	944	109	33	240	80	1406
114	880	97	28	222	75	1302
120	815	73	21	202	68	1179
132	660	50	15	155	50	930
144	527	50	15	126	38	756
156	427	37	11	95	32	602
168	334	29	9	76	26	474
180	270	29	9	65	22	395
210	186	20	6	46	16	274
240	146	17	5	37	13	218
270	122	15	4	32	12	185
300	113	17	5	30	11	176
330	120	17	5	34	12	188
360	120	17	5	33	12	187
390	56	0	0	11	4	71
420	10	0	0	0	0	10
450	0	0	0	0	0	0

The main diversion spillway is designed to provide optimum attenuation of the drainage basin inflow hydrograph, thus minimizing offsite impacts from flood flows across the reclaimed site. The design is also an economic trade-off between providing additional upper tailings impoundment storage volume and excavating an excessively large spillway--that is, in excess of the rock required for selective riprap armouring for the rest of the site. The general arrangement of such a spillway is shown in Figure 5.5-11. The first 120 feet of the spillway invert is horizontal at 6692 feet msl and the remaining 250 feet has a bottom slope of 6%, which is steeper than the critical slope, thus causing the flow to be controlled at station 1+20. The flow profile upstream of the control station will be a backwater curve and that downstream of the control will be a drawdown curve. Station 0+00 is at location 607,600 north, 2,638,120 east, and station 3+70 is at location 607,250 north, 2,638,000 east, at the runout as shown in Figure 5.5-12 and the channel alignment is straight with 1:1 sideslopes.

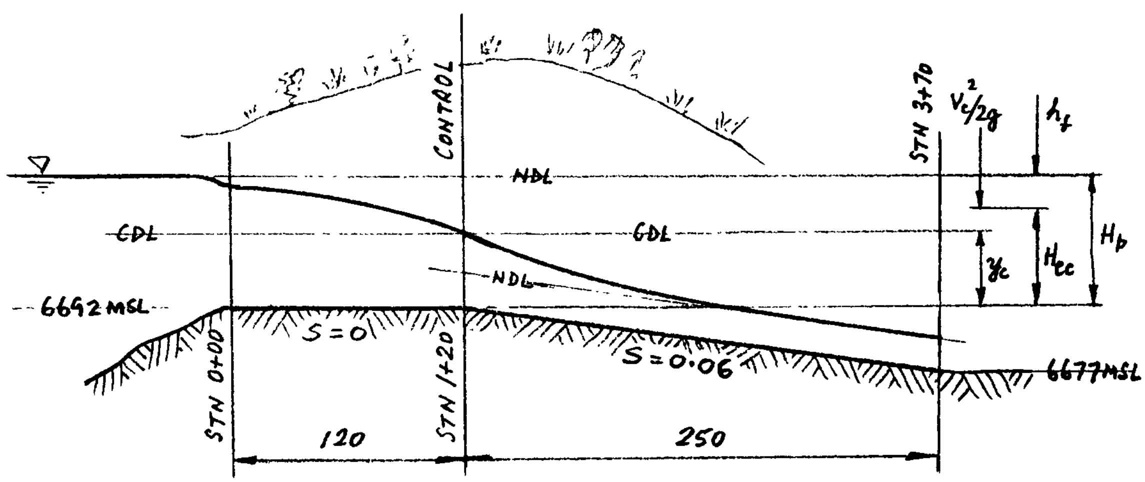


FIGURE 5.5-11 - CROSS SECTION OF MAIN DIVERSION SPILLWAY

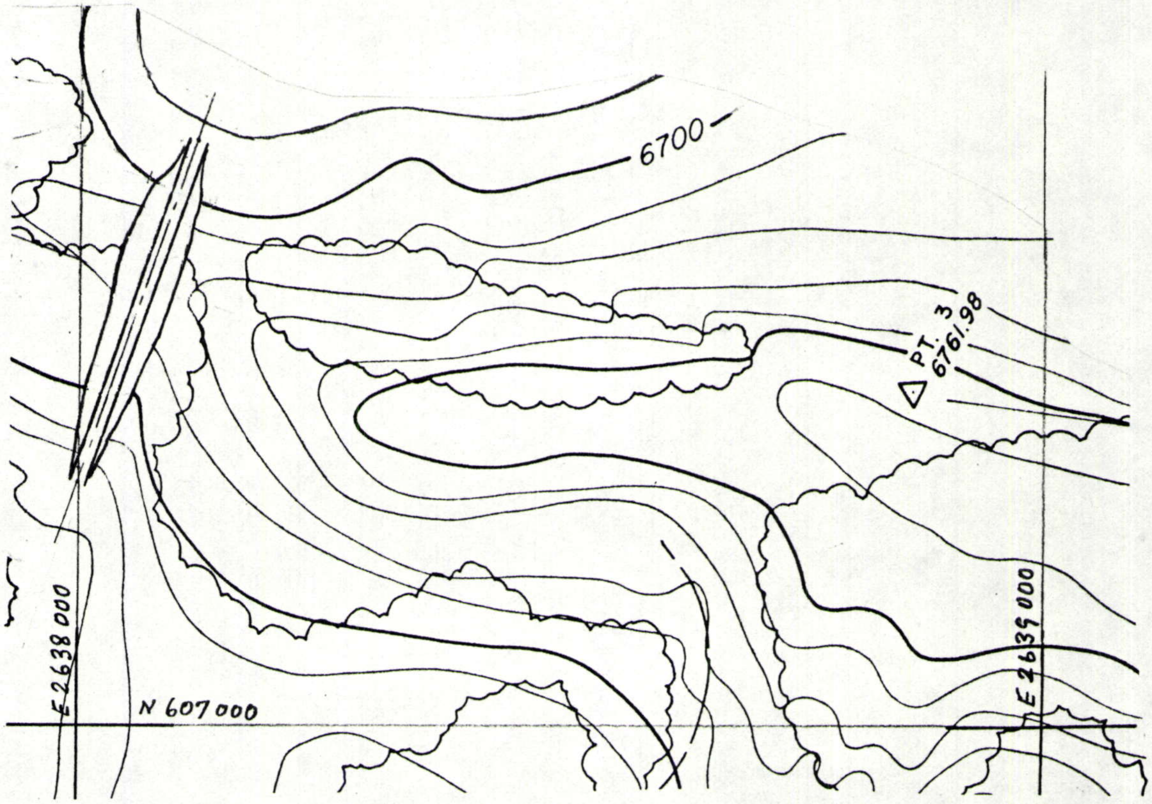


FIGURE 5.5-12 - LOCATION OF MAIN DIVERSION SPILLWAY

The hydraulics of broad crested spillways with a control section can be analyzed with the aid of several nomographs developed by the Soil Conservation Service (1968) and a few simple equations. Analysis procedures are used to define the discharge in the spillway for a given reservoir head, H_p.

When flow occurs across the crest of a spillway with a control, critical depth and critical velocity will occur at the control section. The sum of the velocity head, $V_c^2/2g$, and depth of flow, y_c, at this point is the total critical head, H_{ec}, or

$$H_{ec} = y_c + \frac{V_c^2}{2g} \quad \text{...(19)}$$

where the subscript c refers to critical flow and g is the gravitational constant. The total energy head for any flow is given by

$$H_e = E = y + \frac{V^2}{2g}$$

or in terms of discharge Q and cross sectional area A,

$$E = y + \frac{Q^2}{2gA^2}$$

Minimum energy occurs at critical depth and when the rate of change of energy with respect to depth is zero, that is when

$$\frac{dE}{dy} = 1 - \frac{Q^2}{gA^3} \cdot \frac{dA}{dy} = 0 \qquad \text{or} \qquad \frac{Q^2}{gA_c^2} = \frac{A_c}{dA/dy} \text{.....................(20)}$$

For a rectangular cross section, the critical depth is given in terms of discharge by $y_{c,r} = (q^2/g)^{1/3}$ where q is discharge per unit width and the subscript r refers to a rectangular channel. For a channel of bottom width b and discharge Q, $y_{c,r}$ becomes

$$y_{c,r} = \left(\frac{Q^2}{b^2 g}\right)^{1/3} \text{..(21)}$$

From equations (19) and (21) it can be shown that

$$H_{ec,r} = \frac{3}{2} y_{c,r}$$

hence, (19) can be solved for Q_r in terms of $H_{ec,r}$ and b to yield

$$Q_r = \left(\frac{2}{3}\right)^{3/2} g^{1/2} H_{ec,r}^{3/2} b \text{......................................(22)}$$

Thus, Q_r is uniquely related to $H_{ec,r}$.

The relationship between Q and H_{ec} for a trapezoidal section is more complex than for a rectangular channel and its solution is implicit. However, a simple approximation can be made based on the premise that the ratio of discharges of two trapezoidal channels with equal critical specific energy heads is equal to the ratio of their average widths (SCS, 1968), or

$$\frac{Q}{Q'} = \frac{b + zy_c}{b' + z'y_c'} \qquad (23)$$

When the approximation is based on a rectangular channel ($z' = 0$) of width, $b' = 100$, then equation (23) may be written

$$Q = \frac{(b + zy_c)Q'}{100}$$

and, if the assumption is made that $y_c \simeq y_c'$, then

$$y_c = \frac{2}{3}H_{ec}$$

and

$$Q = \frac{1.5b + zH_{ec}}{150} \qquad (24)$$

Methods are available to determine the approximate error in the use of equation (24), but they were not used because the errors were small and tend to provide an additional degree of safety because their use results in higher flow rates. Another larger degree of safety was incorporated in the design by using a value of 0.04 for Manning's n, the channel roughness factor. Excavation methods will provide a much smoother surface.

It now remains to relate H_{ec} to H_p in order to develop a stage-discharge relationship. The Soil Conservation Service (1968) has computed numerous backwater profiles and developed nomographs of H_p versus H_{ec} for a standard reference section of b = 100 feet, z = 2, and n = 0.04 for similar spillways to the proposed design. These nomographs were used together with the foregoing equations for many tentative channel designs. The stage-discharge curve for the selected design is shown in Table 5.5-18.

Outflow from the reclaimed upper tailings impoundment "reservoir" (drainage basin) is controlled by the spillway at a rate proportional to the head above the inlet. At the beginning of inflow from a storm, the head above

TABLE 5.5-18 - SPILLWAY STAGE-DISCHARGE CURVE

(1) H_p	(2) H_{ec}	(3) Factor	(4) New H_{ec}	(5) Q_r	(6) Q
0	0	0	0	0	0
1	0.615	1.0015	0.616	149.3	18.5
2	1.480	1.0032	1.485	559.0	72.6
3	2.420	1.005	2.432	1171.5	159.6
4	3.380	1.007	3.404	1939.9	276.8
5	4.340	1.009	4.379	2830.4	422.3
6	5.320	1.011	5.379	3853.4	600.6
7	6.310	1.012	6.386	4984.6	810.4

Notes on Table 5.5-18:

(1) Feet head above spillway crest (or, stage-feet in reservoir).

(2) Total energy head for H_p feet from SCS (1968) nomograph for z = 2 (in feet).

(3) Sideslope correction factor from SCS (1968) nomograph for z = 1.

(4) Total energy head for H_p feet and z = 1 (in feet).

(5) Discharge in standard 100-foot wide rectangular spillway from equation (22) (in cfs).

(6) Corrected discharge for proposed design of trapezoidal section from equation (24) (in cfs). b = 12 feet, z = 1, and n = 0.04.

the inlet is small and the inflow rate exceeds the outflow rate. Reservoir storage increases, increasing the depth in the reservoir, thereby increasing the outflow rate. This process continues until the outflow rate equals the inflow rate, occurring after the peak of the inflow hydrograph.

The change in storage over a time interval t_1 to t_2 is

$$S_2 - S_1 = \Delta S = \left(\frac{I_2 + I_1}{2}\right)\Delta t - \left(\frac{O_2 + O_1}{2}\right)\Delta t \quad \text{.....................(25)}$$

where S_2 is the storage at time t_2 and S_1 is the storage at time t_1. I is the inflow rate, O is the outflow rate, and Δt is the time interval over which ΔS is measured (t_2-t_1). The unknowns in this equation are S_2 and O_2 and, since S and O are both functions of stage, H_p, they are functionally related and allow a unique solution for O_2. Successive iterations are required because the solution is not explicit. The following procedure was used to define the spillway outflow hydrograph by this numerical flood routing method:

(1) Assume that $O_2 = O_1$

(2) Calculate ΔS from equation (25)

(3) Knowing S_1, $S_2 = S_1 + \Delta S$

(4) From stage-storage curve (Table 5.5-16, columns 5 and 6), determine H_2 from S_2

(5) Calculate new O_2 from the stage-discharge curve (Table 5.5-18, columns 1 and 6)

(6) Repeat above steps until O_2 remains unchanged.

Results of the routing are shown in Figure 5.5-13 which compares the outflow hydrograph with the inflow hydrograph (of column 7, Table 5.5-17). The peak offsite flow has been reduced by 68% to 608 cfs, occurring 156 minutes after the beginning of the 6-hour local-storm PMP. The stage-feet at 608 cfs discharge is 6.08 feet giving a maximum possible reservoir storage level of 6698.08 feet msl.

In addition to routing the PMP storm, inflow hydrographs for the 6-hour 100-year and 1000-year storms were also developed and routed through the reservoir and spillway. Maximum spillway discharge rates were 45 cfs and 111 cfs and the maximum storage levels were 6693.5 feet msl and 6694.5 feet, respectively, for the 100-year and 1000-year storms. The results are graphically compared in Figure 5.5-14, which shows the stage-discharge and stage-storage curves.

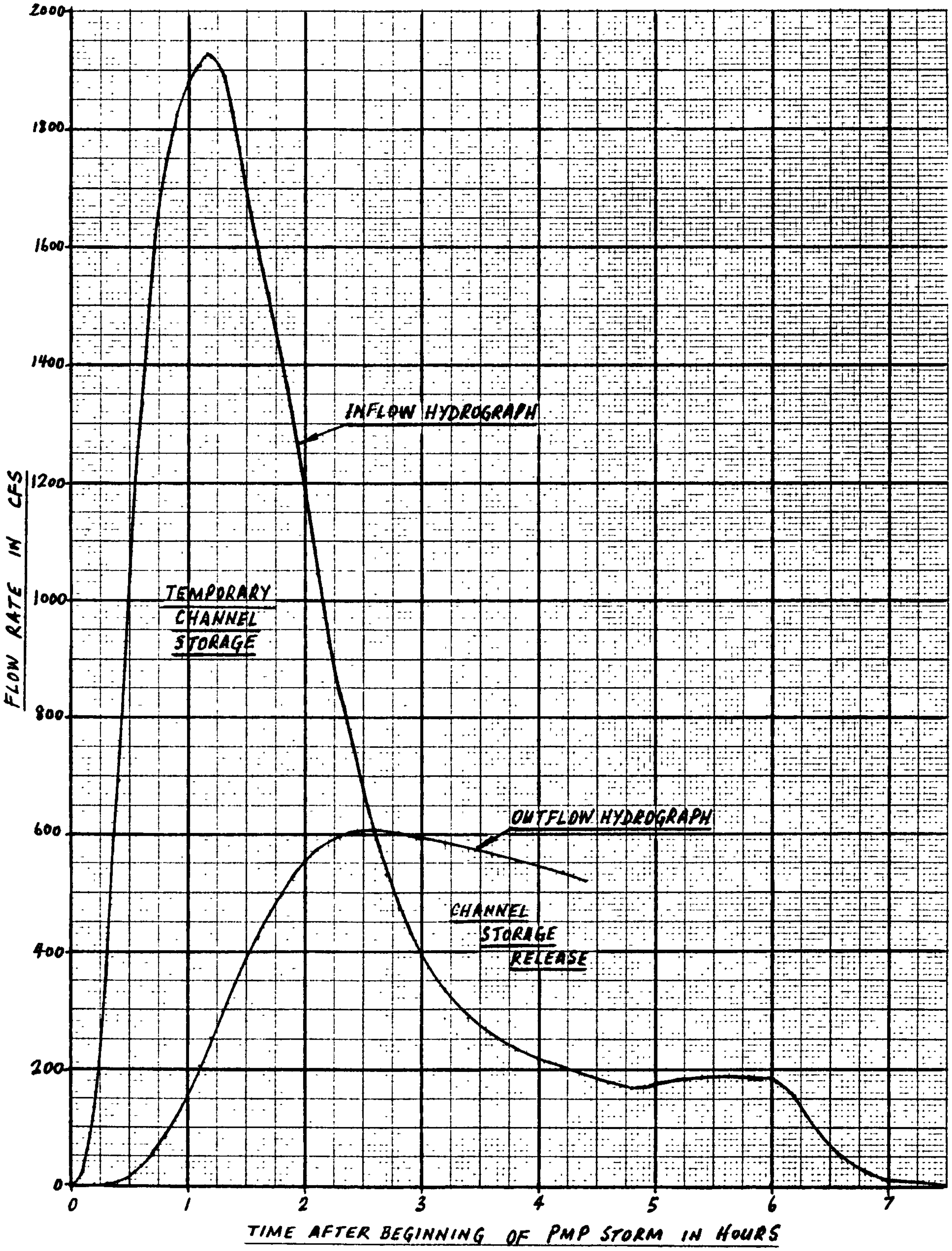


FIGURE 5.5-13 SPILLWAY INFLOW AND OUTFLOW HYDROGRAPHS

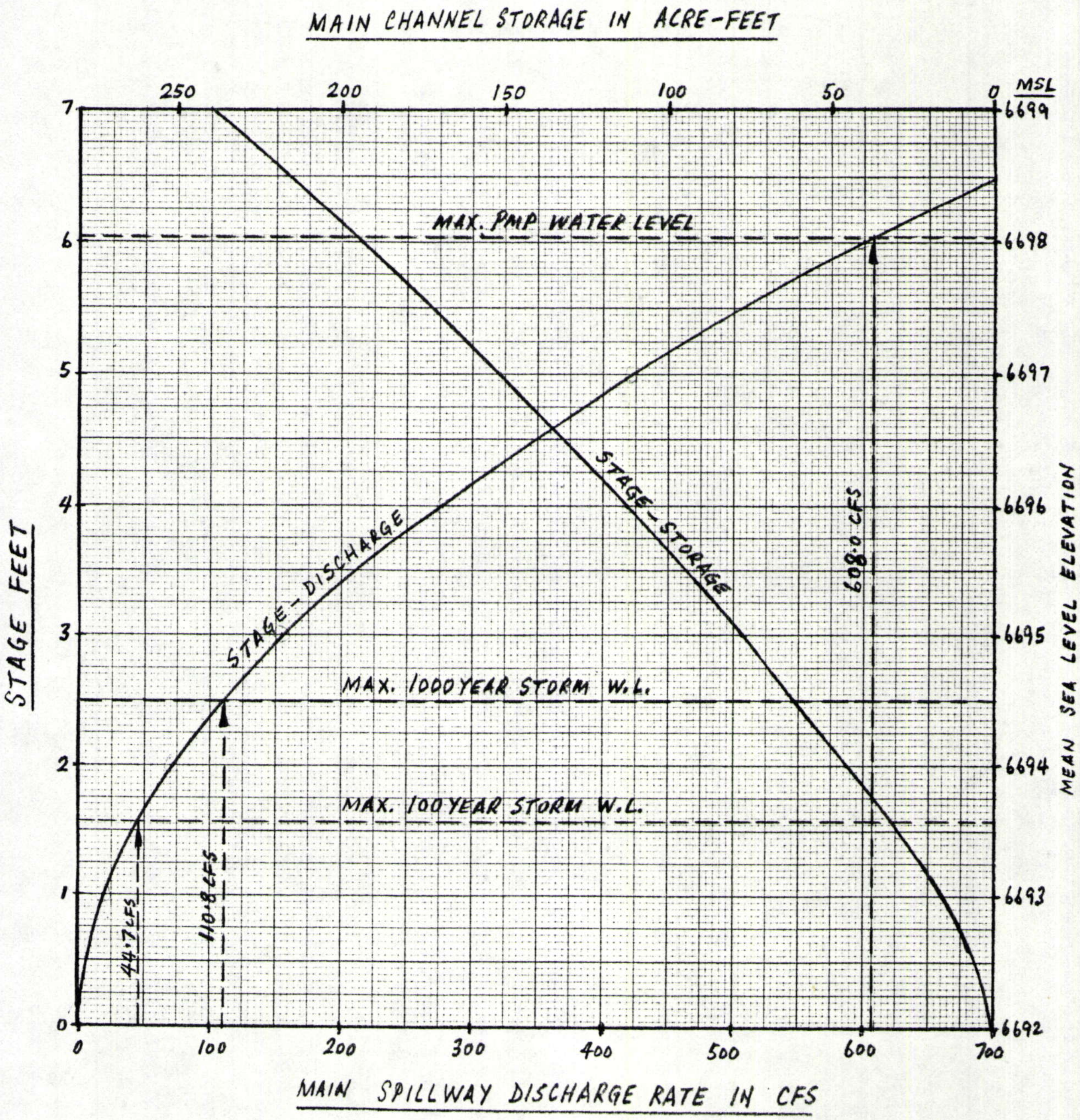


FIGURE 5.5-14 STAGE CURVES SHOWING STORM COMPARISONS

A secondary, riprap lined channel will be used to divert concentrated runoff from the upper tailings impoundment northeast watershed to prevent eroding the cover on the east side of the impoundment. The SCS curve number method and SCSHYDRO code were also used to determine peak flows from the sub-watersheds contributing flow to this drainage channel. As shown in Figure 5.5-10, sub-watershed A-3a only contributes flow at station 0+00 of the channel and the combined flows from sub-watersheds A-3a, A-3b, and A-3c contribute flow to the channel at station 8+00. The peak flows were found to be 93 cfs and 206 cfs and their times to peak were 0.62 hours and 0.75 hours, respectively, for stations 0+00 and 8+00. The adequacy of the proposed design to convey these peak PMF flow rates was determined using the Manning and continuity equations:

$$V = \frac{1.486}{n} R^{2/3} S^{1/2} \qquad (26)$$

and

$$Q = AV \qquad (27)$$

where V is the flow velocity (ft/sec), R the hydraulic radius (ft), S the slope (ft/ft), n the channel roughness factor (dimensionless), Q the discharge rate (ft^3/sec), and A is the channel flow area (ft^2). The flow was then classified using the Froude number, which is defined by the equation

$$F = \frac{V}{(gy)^{1/2}} \qquad (28)$$

where F is a dimensionless number, V is the flow velocity (ft/sec), g the gravitational constant, and y the average depth of flow (ft). Critical flow occurs when $F = 1$. If $F < 1$, the flow is said to be subcritical or tranquil, and if $F > 1$, it is classified as supercritical or turbulent.

Two main design procedures are used for insuring the stability of erodible channels. One procedure is based on the limiting velocity concept and the other on the limiting tractive force concept. When using the limiting velocity concept, the channel is simply sized so that it has adequate capacity and that the velocity does not exceed some permissible value for the channel lining. The channel will be constructed primarily in soil (silty sand) with some rock sections. If a limiting velocity, V_p, of 2.5 ft/sec is assumed and a Manning's n of 0.024 and a slope of 2.5% are used, equation (26) yields

$$V_p = 2.5 \geq \frac{1.486}{0.024} R^{2/3} (0.025)^{1/2}$$

from which $R \leq 0.125$ feet for diversion to be stable. Equation (27) is then used to design for capacity

$$q_p = AV_p \qquad \text{or} \qquad A = \frac{q_p}{V_p} = \frac{206 \text{ cfs}}{2.5 \text{ ft/sec}} = 82.4 \text{ ft}^2$$

A channel more than 330 feet wide is required to meet these conditions ($R \leq 0.125$, $A \geq 82.4$). Therefore, a riprap lined channel is needed. The tractive force concept will be used to design the riprap size.

Following the procedures of Stevens and Simons (1971) and Simons and Senturk (1977) it can be shown that the safety factor, SF, for a given flow situation on the bed of a channel is given by

$$SF_b = \frac{\cos\theta \tan\phi}{\sin\theta + \eta_b \tan\phi} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (29)$$

where θ is the slope of the channel bottom, ϕ is the permissible angle of repose of the rock, and η_b is a stability parameter that can be shown to be

$$\eta_b = \frac{21\,\tau}{\gamma(SG-1)D} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (30)$$

where SG is the specific gravity of the rock particles, D is the average particle size (ft), γ is the specific weight of water (lb/ft^3), and τ is the average shear force (lb/ft^2) on the rock particles given by dS, where d is the depth of flow (ft) and S is the channel bottom slope (ft/ft). Also, Manning's n can be found from

$$n = 0.0395(D_{50})^{1/6} \quad \text{.......} (31)$$

where D_{50} is the average particle diameter in feet. For simplicity, a wide channel is assumed and Manning's equation is used to determine riprap size needed for a stable channel by a trial and error method.

By trying a D_{50} riprap of 0.4 feet, Manning's n can be found to be $n = 0.0395(0.4)^{1/6} = 0.0339$, and from Manning's equation

$$d = \left(\frac{n\,Q}{1.486bS^{1/2}}\right)^{3/5} = \left(\frac{(0.0339)(93)}{(1.486)(10)(0.025)^{1/2}}\right)^{3/5} = 1.193 \text{ ft. depth}$$

required to convey 93 cfs in a 10-foot wide channel. To check for stability $\tau = \gamma dS = (62.4)(1.193)(0.025) = 1.86\ lb/ft^2$ shear force, and

$$\eta_b = \frac{21\,\tau}{\gamma(SG-1)D_{50}} = \frac{(21)(1.86)}{(62.4)(2.65-1)(0.4)} = 0.949$$

For a 2.5% bottom slope, $\theta = 1.432°$, and for angular riprap, $\phi = 41.6°$. Hence, the safety factor for the channel bottom is

$$SF_b = \frac{\cos 1.432 \tan 41.6}{\sin 1.432 + 0.949 \tan 41.6} = 1.023$$

Since $SF_b > 1$, the lining selected is stable.

For ease of construction, a trapezoidal channel with 3:1 sideslopes is selected. The flow area for such a channel is

$$A = bd + zd^2$$

Therefore, the velocity of flow is

$$V = \frac{93}{(10)(1.193) + (3)(1.193)^2} = 5.74 \text{ ft/sec}$$

and the Froude number is

$$F = \frac{5.74}{[(32.2)(1.193)]^{1/2}} = 0.926$$

indicating that the flow is tranquil at station 0+00.

Flow conditions at station 8+00 were analyzed in the same manner and it was found that SF_b = 1.126, V = 6.72 ft/sec, and F = 1.082 when the same size riprap was used and a bottom width of 22 feet was chosen to convey the peak PMF flow of 206 cfs. It is therefore reasonable to assume that a gradually tapering channel from 10 feet at station 0+00 to 22 feet at station 8+00 will be adequate to convey peak PMF flows. No additional runoff enters the channel after station 8+00 and its width therefore remains constant until its bottom slope drops to zero at approximately station 13+20 at 608,380 north, 2,639,200 east. At this point the channel bottom width will increase until it is 100 feet wide and its 3:1 sideslopes will flatten until they are non-existent at the brink of the main diversion channel 80 feet away. The median rock size for this riprap apron will be the same as used in the channel itself to dissipate the concentrated flow energy before it enters the main channel.

Using the same tractive force method it was found that riprap is also needed on the 10:1 slope into the main diversion channel. The sloping length of this bank is 80.4 feet to drop the flow from 6,693 feet msl to 6,685 feet msl. Although the main channel will be flooded when flow from this secondary channel enters, riprap was sized as though the main channel was dry for additional safety. With a medium riprap of 0.5 feet and the 206 cfs spread out over 100-foot width, it was found that SF_b = 1.06, V = 6.3 ft/sec, and F = 1.95 indicating turbulent flow conditions.

It is also necessary to check the stability of the secondary diversion channel banks (3:1 sideslopes) because the drag forces are not parallel with the downslope gravitational forces. The solutions of the equations describing the safety factor for this case have been given by Stevens and Simons (1971) and Simons and Senturk (1977) as

$$SF = \frac{\cos\alpha \tan\phi}{\sin\alpha \cos\beta + \eta' \tan\phi}$$

where $$\beta = \tan^{-1}\left(\frac{\cos\alpha}{\frac{2\sin}{\eta\tan} + \sin\lambda}\right)$$

$$\eta = \frac{21\,\tau_{max}}{\gamma(SG-1)D_{50}}$$

and $$\eta' = \eta\left(\frac{1 + \sin(\lambda + \beta)}{2}\right)$$

where τ_{max} is the maximum shear on the channel bank. For trapezoidal channels with 3:1 sideslopes, the maximum tractive force on the channel banks is $0.8\gamma dS$ (Lane, 1953).

Therefore, choosing the same D_{50} as for the channel bed

$$\tau_{max} = (0.8)(62.4)(1.193)(0.025) = 1.489 \text{ lb/ft}^2$$

$$\eta = \frac{(21)(1.489)}{(62.4)(1.65)(0.4)} = 0.759$$

Assuming uniform flow, the streamlines are parallel to the channel bed and

$\lambda = \theta = 1.432°$. Also, for a 3:1 sideslope

$$\alpha = \tan^{-1}(1/3) = 18.44°$$

and $$\beta = \tan^{-1}\left(\frac{\cos 1.432}{\frac{2\sin 18.44}{0.759\tan 41.6} + \sin 1.432}\right) \qquad (\phi = 41.6°)$$

$$= 46.055°$$

$$\eta' = 0.759\left(\frac{1 + \sin(1.432 + 46.055)}{2}\right) = 0.659$$

$$SF = \frac{\cos 1.432 \tan 41.6}{\sin 1.432 \cos 46.055 + 0.659 \tan 41.6} = 1.47 > 1$$

Therefore, selected riprap is adequate and design is stable.

All types of stone lining should be laid on a filter blanket of gravel or crushed stone unless the gradation of the natural soil is such that it will not filter up through the stone lining. The following criteria have been established for sizing a filter, based on the size distribution of the riprap and the base material:

(1) $\frac{D_{50}\text{ (filter)}}{D_{50}\text{ (base)}} < 40$ and $\frac{D_{50}\text{ (riprap)}}{D_{50}\text{ (filter)}} < 40$

(2) $5 < \frac{D_{15}\text{ (filter)}}{D_{15}\text{ (base)}} < 40$ and $5 < \frac{D_{15}\text{ (riprap)}}{D_{15}\text{ (filter)}} < 40$

(3) $\frac{D_{15}\text{ (filter)}}{D_{85}\text{ (base)}} < 5$ and $\frac{D_{15}\text{ (riprap)}}{D_{85}\text{ (filter)}} < 5$

Filter thickness should be one-half the thickness of the riprap, but in no case less than six inches.

According to Simons and Senturk (1977) the channel riprap gradation would be as follows:

Size (in)	Size (ft)	% Finer
0.48	0.04	0
2.02	0.17	15
2.40	0.20	20
4.80	0.40	50
8.16	0.68	85
9.60	0.80	100

and the estimated D_{15}, D_{50}, and D_{85} of the bank material are 0.0015 in., 0.0055 in., and 0.02 in., respectively. To check if a filter is needed:

$\frac{D_{15}\text{ (riprap)}}{D_{85}\text{ (base)}} < 5$, that is $\frac{2.02\text{ in.}}{0.02\text{ in.}} = 101 > 5$. Therefore, a filter blanket is needed.

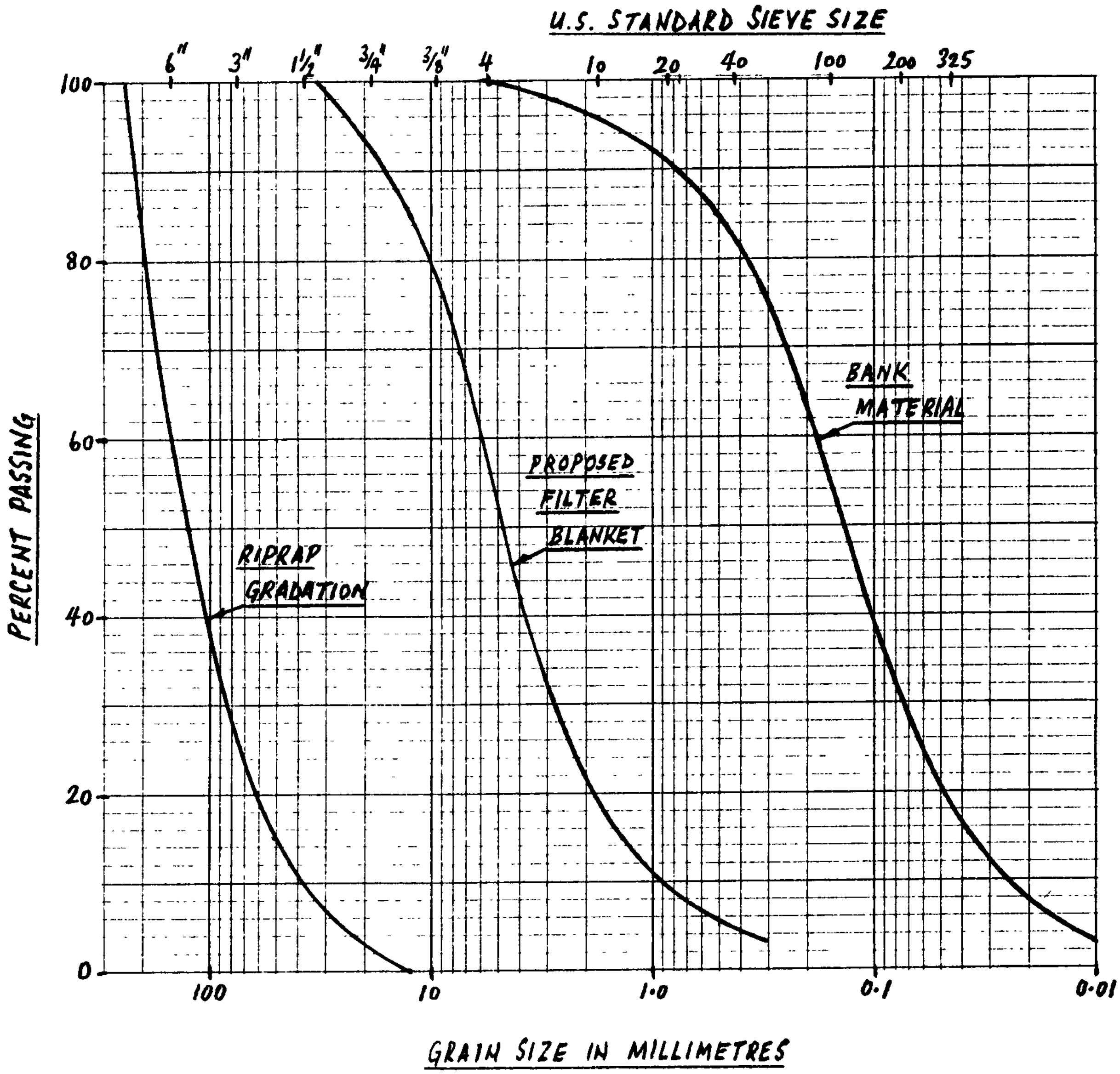


FIGURE 5.5-15 SECONDARY DIVERSION FILTER BLANKET

Figure 5.5-15 shows a proposed gradation of filter blanket in which the D_{15}, D_{50}, and D_{85} are 0.055 in., 3/16 in., and 1/2 in., respectively. The adequacy of the proposed filter blanket can now be checked against the necessary criteria:

	(a) Filter and Base	(b) Riprap and Filter
(1)	$\frac{D_{50}\text{ (filter)}}{D_{50}\text{ (base)}} = \frac{0.1875}{0.0055} = 34.1$	$\frac{D_{50}\text{ (riprap)}}{D_{50}\text{ (filter)}} = \frac{4.80}{0.1875} = 25.6$
(2)	$\frac{D_{15}\text{ (filter)}}{D_{15}\text{ (base)}} = \frac{0.055}{0.0015} = 36.7$	$\frac{D_{15}\text{ (riprap)}}{D_{15}\text{ (filter)}} = \frac{2.02}{0.055} = 36.7$
(3)	$\frac{D_{15}\text{ (filter)}}{D_{85}\text{ (base}} = \frac{0.055}{0.02} = 2.75$	$\frac{D_{15}\text{ (riprap)}}{D_{85}\text{ (filter)}} = \frac{2.02}{0.5} = 4.04$

All three criteria are satisfied for both cases; therefore, proposed filter blanket is satisfactory.

A typical cross section of the channel is shown in Figure 5.5-16.

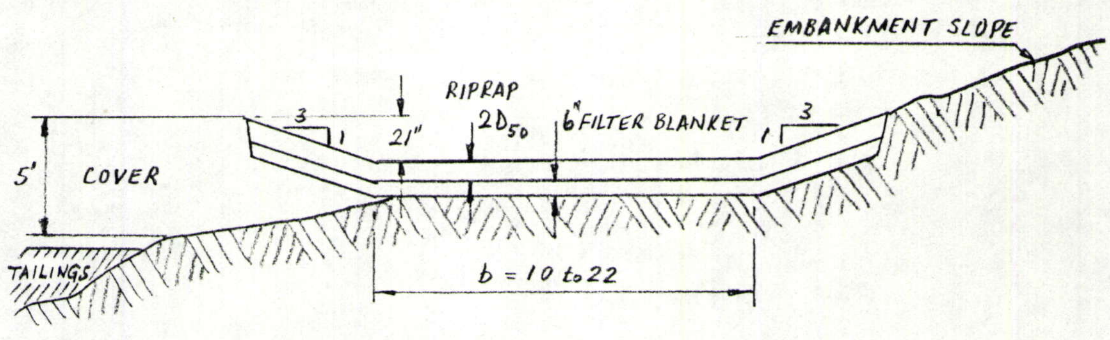


FIGURE 5.5-16 - TYPICAL CROSS SECTION OF SECONDARY DIVERSION

A-4 watershed runoff is directed toward the main channel spillway to avoid the problems associated with allowing it to overtop the upper tailings embankment. To achieve this objective, it is necessary to correctly contour the upper tailings cover material within A-4 watershed as shown in Figure 5.5-10. The peak PMF runoff rate for this watershed's outlet was found to be 151.5 cfs and that entering the "diversion", 1,000 feet upstream (from sub-watersheds A-4c, A-4d, A-4e and the small area of cover material), was found to be 48.1 cfs. The flow at both stations was found to be tranquil and less than 2 ft/sec velocity; therefore, no armouring is required.

As with the upper tailings embankment, no overbank flow of the lower embankment was permitted. Runoff from the smaller C-1 watershed tends to concentrate at the cover south abutment. A small 1,300 foot long diversion channel was therefore used to eliminate this runoff as soon as possible. The channel will be a triangular cross section for the first 900 feet and gradually change to a trapezoidal section from station 9+00 to station 9+50. It will remain trapezoidal until it outlets at station 13+00 located at 607,745 north, 2,636,580 east. Typical sections are shown in Figure 5.5-17 and the channel design is presented in Table 5.5-19 which shows the expected peak PMF flows. The channel will curve very gradually for the first 1,050 feet and thereafter be straight with control provided at station 11+00. Much of the channel will be excavated by ripping and blasting; that is, the triangular sections and below the control. It is doubtful, therefore, that any riprap will be needed, especially since the only "rapid" flow occurs after the control station. The peak inflow to the channel was 70.5 cfs, which occurs 0.61 hours after the beginning of the PMP storm.

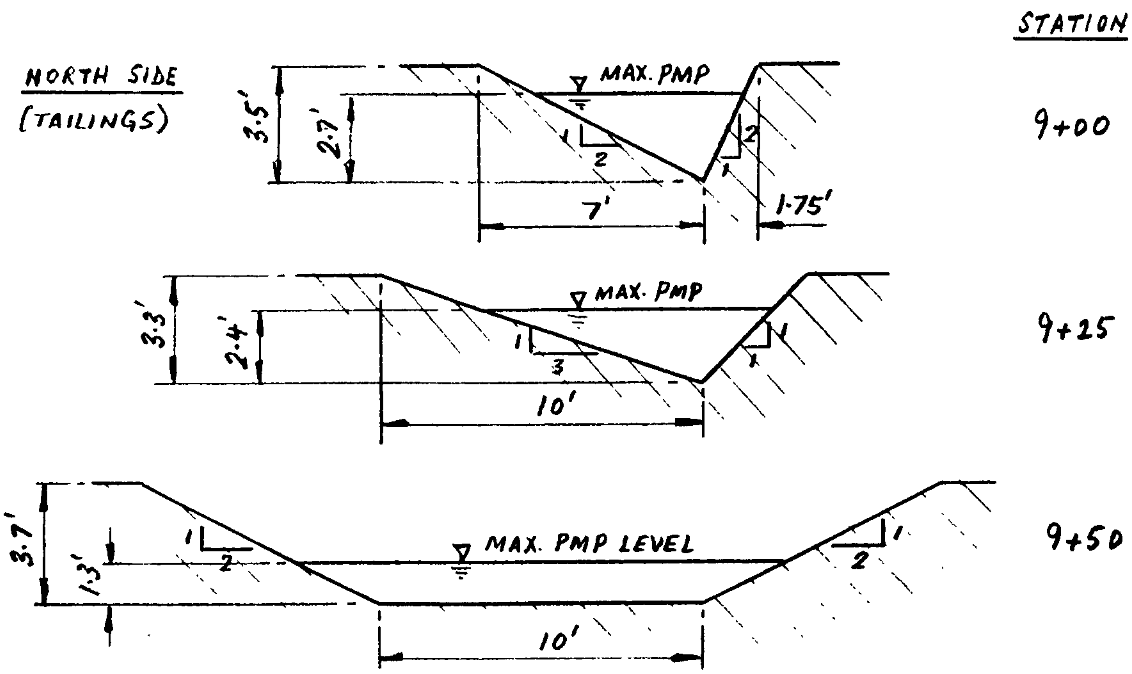


FIGURE 5.5-17 - TYPICAL C-1 DIVERSION CROSS SECTIONS

TABLE 5.5-19 - C-1 DIVERSION DESIGN PARAMETERS

Station	Bank Elevation	Invert Elevation	Bottom Slope	Peak Flow	Flow Depth	Flow Velocity	Froude Number
0+00	6654.2	6652.2	-	9.13	1.164	5.39	0.88
1+00	6651.9	6649.7	0.025	10.80	1.250	5.53	0.87
2+00	6649.5	6647.3	0.024	13.30	1.362	5.74	0.87
3+00	6647.4	6645.0	0.023	17.50	1.522	6.04	0.86
4+00	6645.5	6642.8	0.022	23.97	1.728	6.42	0.86
5+00	6643.8	6640.7	0.021	33.30	1.973	6.85	0.86
6+00	6642.5	6638.7	0.020	42.43	2.203	6.99	0.83
7+00	6641.4	6636.9	0.018	50.80	2.410	7.00	0.79
8+00	6639.5	6635.3	0.016	59.74	2.561	7.29	0.80
9+00	6637.2	6633.7	0.016	70.48	2.725	7.60	0.81
9+25	6636.6	6633.3	0.016	70.48	2.397	6.13	0.70
9+50	6636.6	6633.1	0.010	70.48	1.300	4.30	0.67
10+00	6636.3	6632.6	0.005	70.48	1.300	4.30	0.67
11+00	6635.8	6632.1	Control	70.48	1.021	5.73	1.00
12+00	6630.1	6628.1	0.040	70.48	0.714	8.64	1.80
13+00	6626.0	6624.1	0.040	70.48	0.714	8.64	1.80

The C-2 diversion will be a straight, trapezoidal channel with 2:1 side slopes and 10 ft. bottom width and run perpendicular to the lower tailings embankment from station 0+00 (608920 north, 2635530 east) to station 5+00 (608500 north, 2634270 east). The channel bottom will be level, at 6633 ft. msl, up to the control station at 2+50; thereafter, it will drop at 10% to runout at 6608 ft. msl. The level section shall be constructed with compacted earth and the supercritical section will be excavated in rock, or riprapped where needed.

With an embankment crest of 6638 ft. msl (that is, 5 ft. above the tailings cover "invert") considerable storm runoff attenuation can be achieved. Table 5.5-20 shows the impoundment stage-storage capacity and the channel stage-discharge characteristics and Figure 5.5-18 shows the impoundment inflow and outflow hydrographs--which indicates a 73% PMP storm attenuation. These results were obtained by using the procedures used to design the main channel spillway. In addition, when the 100-year and 1000-year 6-hour local-storms were similarly routed, 3.8 ft. and 2.6 ft. of freeboard was available at the peak storage condition, respectively.

TABLE 5.5-20 C-1 DIVERSION STAGE CURVES

ELEVATION msl	STAGE feet	STORAGE acre-feet	DISCHARGE cfs
6633	0	0	0
6634	1	1.075	4.7
6635	2	4.408	18.5
6636	3	9.921	32.7
6637	4	17.526	54.0
6638	5	26.997	80.7

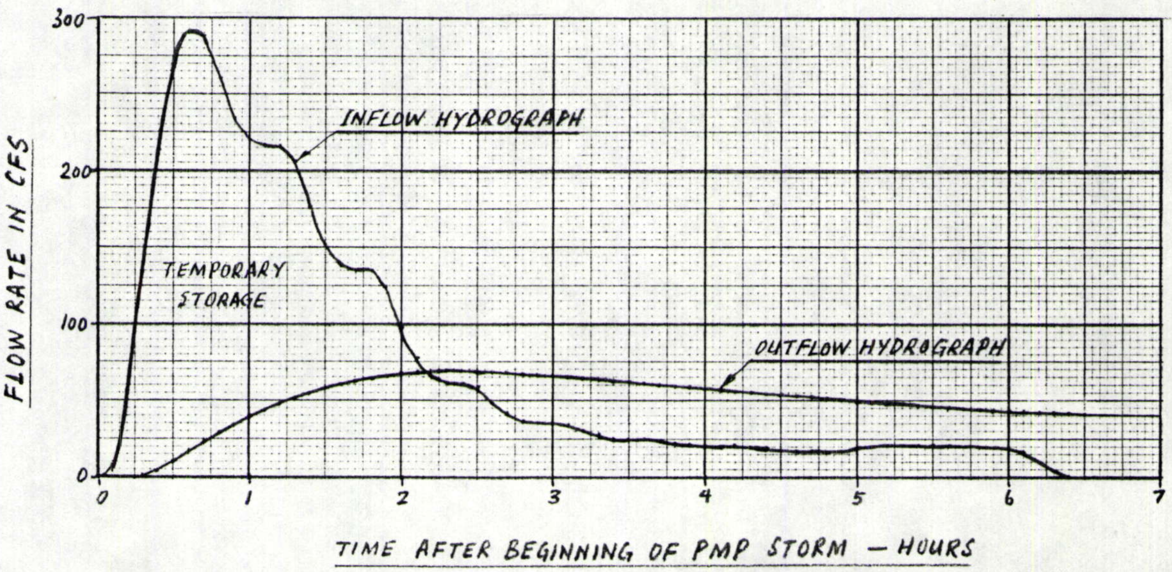


FIGURE 5.5-18 C-2 DIVERSION INFLOW AND OUTFLOW HYDROGRAPHS

Selective armouring will be provided on the embankment downstream faces and at locations around the perimeter of the tailings cover where flow concentrations could cause damage. According to the Army Corps of Engineers (Hydraulic Design Criteria--Sheet 712-1) and the California Department of Highways (Hydraulic Design Series No. 4--Design of Roadside Drainage Channels, Figure 21) riprap is not required on the tailings embankments when there is no overbank flow. However, those steep (2:1 for the upper and 2½:1 for the lower embankment) surfaces do need some protection, since construction methods are not perfect and even small concentrations of runoff may eventually lead to gullying--probably within 1000 years. Also, if the USLE equation is used it can be shown that riprap is needed (References 7 and 8).

The longest slope length on the downstream face of the upper embankment will be 77 ft. at reclamation time. With this slope length and the PMP storm it was determined that the peak flow rate down the embankment (at the bottom) was 0.01 cfs/ft. run. The tractive force method (as used in the design of the secondary diversion riprap) was used to find that a median riprap size of 1½ inches would be stable on this slope under PMF conditions. It was also found that a filter blanket would be needed which has D_{15}, D_{50}, and D_{85} specifications of 0.007 in., 0.05 in., and 0.1 in., respectively. Therefore, the old Keystone-Wallace copper tailings sand could be used. A cross-section of the embankment at reclamation is shown in Figure 5.5-19.

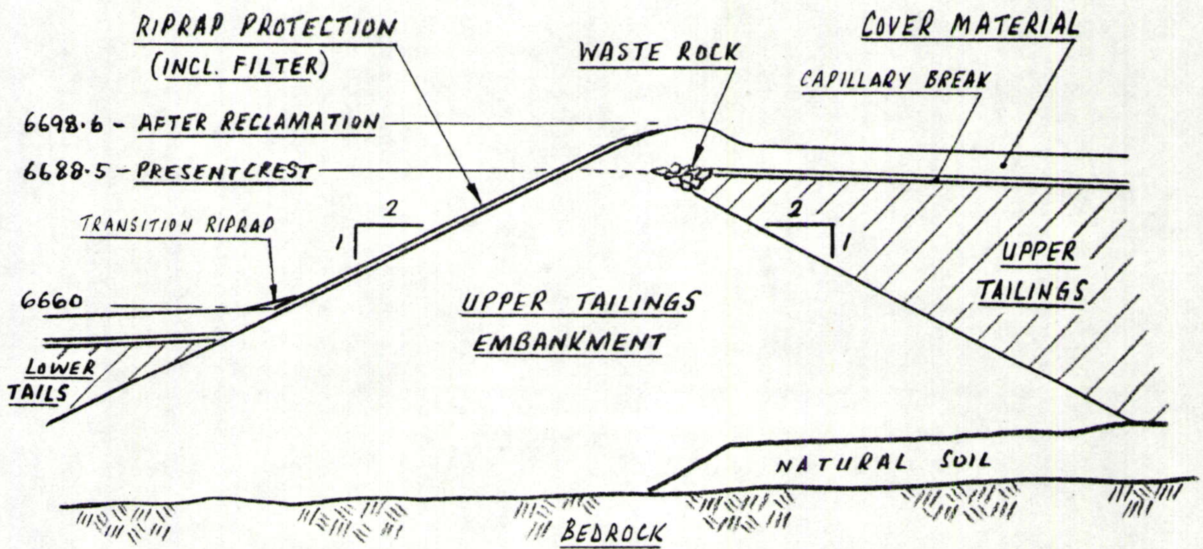


FIGURE 5.5-19 CROSS SECTION OF UPPER DAM AT RECLAMATION

Although the lower tailings embankment downstream slope is less than that of the upper embankment it is subject to a greater concentration of runoff because of the additional slope length (see Figure 5.5-20). However, it was found that the same 1½ inch median riprap and filter would be stable under PMP conditions on the 2½:1 slope. No protection will be used on the 10:1 slope.

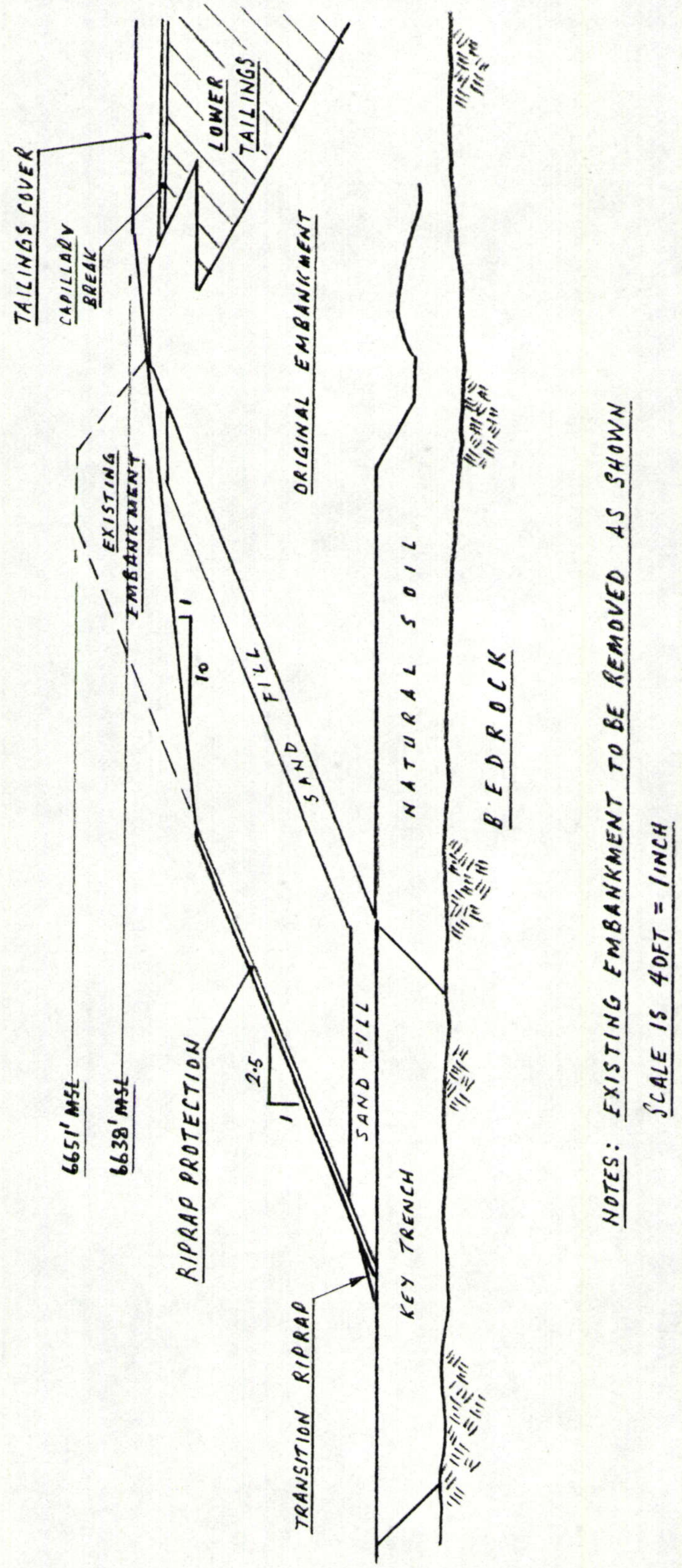


FIGURE 5.5-20 CROSS SECTION OF LOWER DAM AT RECLAMATION

Again, using the curve number method and SCSHYDRO code, flow concentrations in A-3 and A-4 subwatersheds above the upper tailings impoundment cover were found to require riprap armouring. The worst case, subwatershed A-3d, needed several grades of riprap to break up the concentrated flow energy. However, in this particular case, before the riprap is placed, it is necessary to provide additional fill material to lessen the drastic changes in slope as shown in Figure 5.5-21. The tractive force method was used to design the riprap size. Additional riprap protection is also needed to dissipate flow energies in subwatershed A-4a, A-4c, A-4d, and A-4b runouts, although a lesser extent than is required for A-3d.

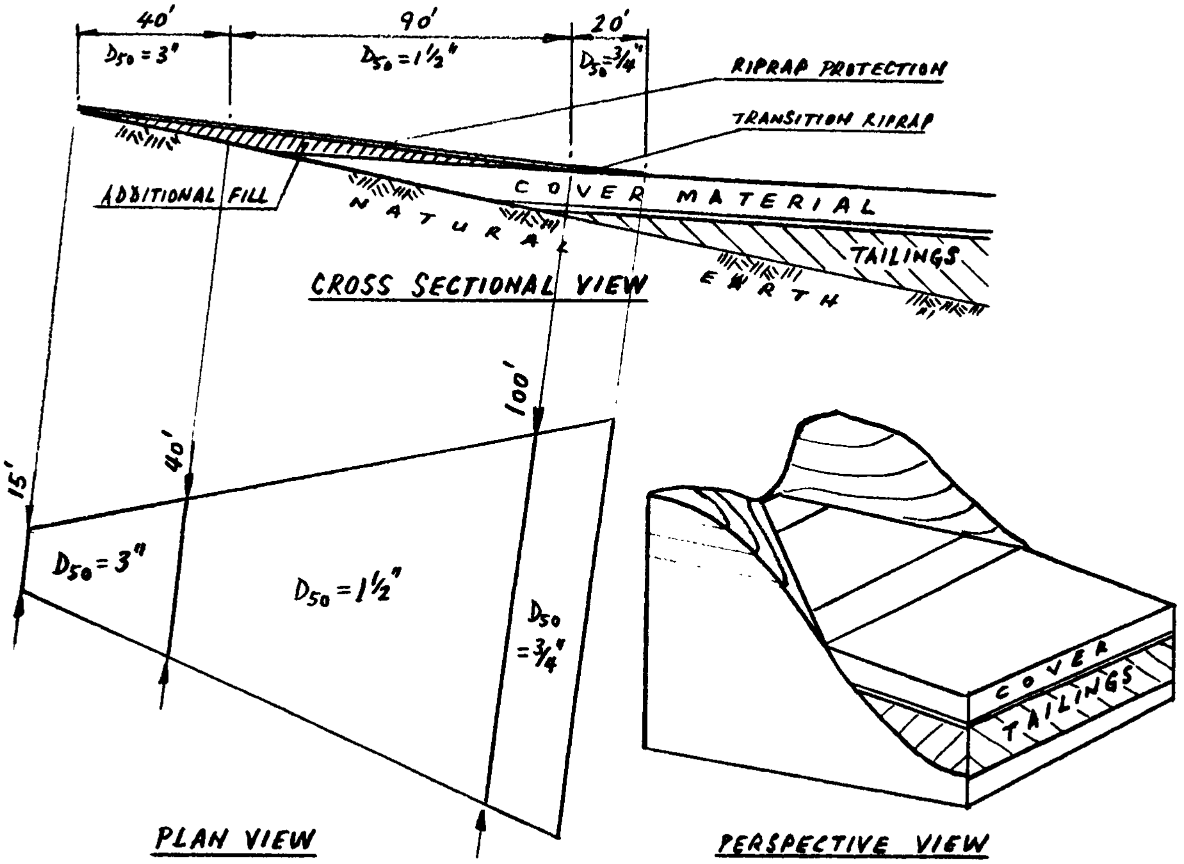


FIGURE 5.5-21 A-3d WATERSHED RIPRAP DESIGN

There are basically only two locations in the lower impoundment that will require such energy dissipating aprons; but, they were not analyzed because of the uncertainty of the condition of this impoundment at reclamation (the upper impoundment is already full, and the cover requirements are already defined). It will be investigated closer to the time of reclamation.

In addition to the concentrated flows mentioned, it is necessary to provide a 10 ft. wide strip of 3/4" (D_{50}) riprap around the rest of the impoundment perimeter. This riprap will protect the cover material at its abutment with the watershed above by providing a transition for sheet flow onto the cover. Thus eliminating any possibility of "undercutting" the cover. Again, the tractive force method was used to size the riprap and the idea is illustrated in Figure 5.5-22.

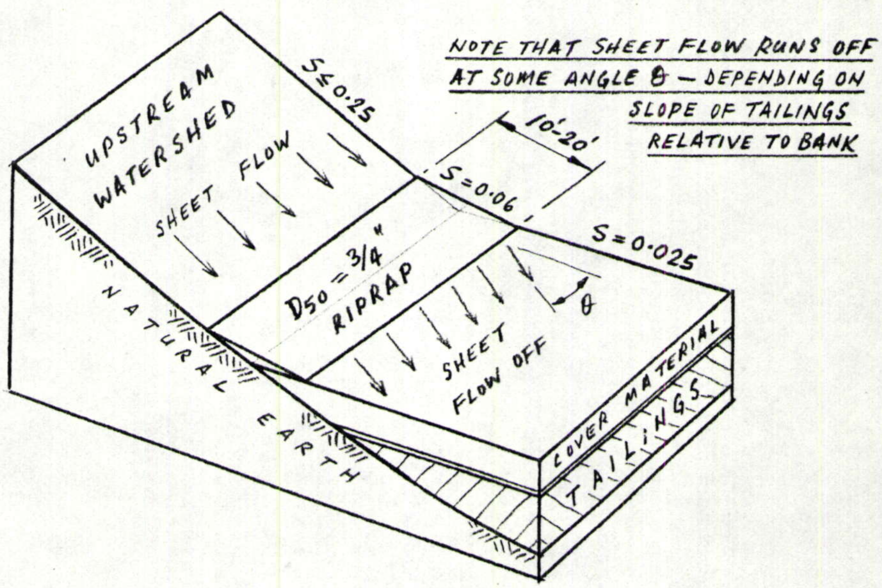


FIGURE 5.5-22 RIPRAP FOR FLOW TRANSITION

Sedimentation

Provision has been made in the main diversion channel to accept all expected sedimentation from the watershed above the upper embankment and in the lower impoundment from its much smaller watershed. It is unlikely that either "structure" will have a silting problem after the post-construction stabilization period. Hence, no offsite sedimentation from the reclaimed site is expected. (References 7, 8)

Revegetation

The disturbed plantsite and tailings cover will be vegetated as the final step in reclamation. A BLM approved seed mixture will be used and spread with fertilizer by hydroseeding the mixture directly on the surface of the newly emplaced substitute topsoil. After the seed and fertilizer have been applied, native grass hay-mulch will be applied by hydroseeding at the rate of 2000 lb. per acre. The seed, fertilizer, and mulch will all have tackifiers added to the hydroseed mix to ensure proper seed-soil contact and to prevent erosion until acceptable stands of vegetation have been established.

The exact soil nutrient and amendment requirements will be determined after the substitute topsoil has been emplaced. Representative samples will be collected and submitted to an independent laboratory to determine the existing concentrations of key nutrients (primarily nitrogen, phosphorus, and potassium). When the laboratory results have been received and assessed, the required amounts of fertilizer will be applied.

Implementation Schedule

RAMC needs to keep the acid circuit option open. It would therefore be unwise to start to reclaim the presently full upper tailings impoundment for two reasons: if the acid circuit option was taken, the upper tailings impoundment would be needed for remaining carbonate leach ore because the lower impoundment would have to be used for acid tailings; and secondly, if the acid circuit option was taken, a thin layer of acid tailings would be used to put a final "cap" on the alkaline tailings in the upper impoundment before it was reclaimed. The beneficial effects of doing the latter are considerable, as determined by an acid test in June, 1984. A 25 gpl H_2SO_4 solution was allowed to set on top of a sample of alkaline tailing and, within one week, a 2 in. layer of $CaSO_4$ (gypsum) formed on the top part of the tailing sample. In addition, a thin layer of ferric sulphate formed on top of the gypsum and approximately 1/2 in. layer of ferric hydroxide formed below the gypsum. The results are shown diagrammatically below. If NRC grants RAMC approval to use the method on a large scale, RAMC will investigate the matter further for its economic viability. Leaving the upper tailings "bare" should not present any dusting problem because RAMC has 100% beach sprinkling protection already in place in this impoundment.

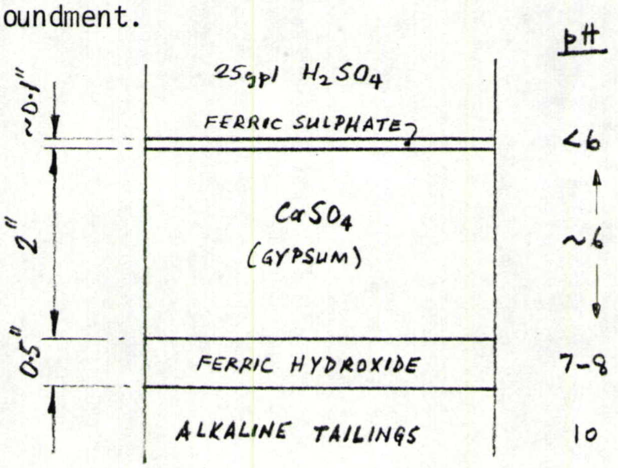


FIGURE 5.5-23 ACID TEST ON ALKALINE TAILINGS

However, pre-closure reclamation work which can be done would be to excavate the main diversion spillway; the various grades of riprap and filter blanket materials could be sized in RAMC's crushing plant and stockpiled for later use. In addition, if an agreement can be worked out with the owners, a substantial amount of Keystone-Wallace copper sand tailings could be hauled to the plantsite and stockpiled for the capillary break and filter media. Nevertheless, it may be unwise to tackle this work prematurely because newer or better ideas may emerge which may render the task redundant; therefore, it is suggested that this phase of the reclamation project be left alone until, at the most, three to four months before final plant closure--and definately not before any decision is made to cease operations permanently. The following is a general outline of the proposed implementation schedule.

Within three months prior to mill shutdown:

- dry out upper tailings and pond, remove sprinkling system.
- close mine down--remove mobile equipment, pumps, etc. (decontaminate for re-sale), plug main haulage from mine shaft.
- mill out ore stockpile and cleanup ore storage pad.
- excavate spillway and crush riprap and filter media in crushing plant.
- haul capillary-break sand from Keystone-Wallace and stockpile.
- haul waste rock from storage area below the toe of Bisco Lake to the waste rock pile.
- re-route recovery well (#OW-UT9) discharge to lower tailings pond (this may be done earlier).

The mill will be shut down after all ore from the storage pad has been cleaned up; possibly, other contaminated earthen materials will be run through the mill circuit also for disposal in the lower tailings impoundment. Then, the following sequence of work will be undertaken (not necessarily in the order shown--some tasks, of course, will run concurrently):

- remove hoisting equipment from mine shaft headframe, decontaminate (where required) and store for resale. Dismantle headframe, decontaminate, and store for resale.
- install sand slurrying system at convenient location to the upper tailings and emplace six-inch thick capillary-break. (~ three months).
- close-down mill operation--cleaning out ore bins, ball mill, pachucas, thickeners, autoclaves, and all tanks, etc. Remove all machinery and equipment--decontaminate and make ready for resale or disposal.
- breakup foundations and concrete for disposal.
- all equipment and materials which can not be decontaminated will be disposed of in the mine shaft before it is capped with concrete. If there are excess contaminated materials, such as soils, etc., that will not fit into the mine shaft with the contaminated equipment, they will be disposed of in the lower impoundment.
- when the plantsite is cleared of all buildings and foundations (perhaps, with the exception of the main administration building and the environmental laboratory) the site will be cleaned up to 40 CFR 192 criteria in accordance with Regulatory Guide 4.14.
- clear proposed tailings cover material borrow area of all topsoil and plantlife--store for later reuse. Move slurrying plant to borrow area and emplace cover material onto upper tailings impound-

ment capillary-break. (~10 to 11 months)

- dry out lower impoundment by sprinkling and thus make ready for emplacement of sand blanket capillary-break and cover material.
- raise upper tailings embankment to 6698.6 ft. msl as shown in Figure 5.5-19 and ensure rest of impoundment "embankment" is raised to 6693 ft. msl.

Approximately fourteen months after mill-closure, the upper tailings impoundment covering will be complete (with the exception of the final cover next to the embankment), and the following sequence of work will then be undertaken:

- move slurrying plant to convenient location and emplace capillary-break sand blanket onto lower tailings. (~3 months).
- re-install slurrying plant back at borrow area and emplace cover material onto lower tailings sand blanket. (~9 to 10 months)
- re-work remaining clean material in the plant area below Bisco Lake embankment to approximately the contours shown in Figure 5.5-10 to make the "connection" with the new spillway.
- remove Bisco Lake embankment (the lake ought to be already dried up) earthen material for temporary stockpiling. Remove the lower waste rock, which was initially used in the construction of this embankment, and dispose of on waste rock pile. The clean earthen material will then be used to cover the waste rock pile to about one-foot thickness--if there is a material shortage, the difference will come from the north bank of the operational flood diversion channel.
- remove other water treatment pond embankments and cleanup contaminated materials for disposal into mine shaft or lower tailings

impoundment (before its covering is completed). Re-grade areas after clean-up.

- remove contaminated, dried-out, materials from Bisco Lake for disposal into mine shaft or lower impoundment.
- remove triple culvert spillway from the upper tailings embankment and re-fill and re-compact embankment at this location.
- excavate secondary diversion channel on east side of reclaimed upper tailings impoundment as shown in Figure 5.5-16. Emplace six-inch thick filter-bed and then place approximately ten-inch thick riprap armouring. Work on the energy dissipator apron and overbank (into the main channel) armouring will be done concurrently. Riprap will be placed by end-dumping and worked into shape with a bulldozer--as will be all other riprap.
- areas above the upper tailings impoundment that are impacted by concentrated flood flows, such as from subwatershed A-3d, will be protected with riprap as shown in Figure 5.5-21. Other areas above the tailings cover that are only subjected to sheet flow impacts will be protected with 3/4 inch (D_{50}) riprap as shown in Figure 5.5-22.

After approximately another thirteen months--or, 27 months after mill-closure--the cover materials will have been placed to form the main southwestern-sloping cover slopes. The following sequence of work will then be undertaken:

- remove slurrying plant to a convenient area between the two tailings embankments (upper and lower) and "cut-down" lower tailings embankment as shown in Figure 5.5-20--the materials will be used to form the contours and cover slopes down from the upstream faces of these embankments as shown in Figure 5.5-10 (the capillary-breaks should first have been installed, of course).

- excavate C-1 diversion channel by bulldozer ripping and/or blasting on the south side of the lower tailing cover and to runout at Station 13+00.
- excavate C-2 diversion channel by the same methods.
- install rip-rap protection for water concentration areas above lower tailings cover and emplace 3/4-inch riprap "transition" protection around lower tailings cover perimeter which is impacted by sheet flow from upstream watersheds and not already protected from concentrated flows.
- remove environmental laboratory and administration building and re-contour ground. (if not completed already).
- install permanent concrete cap on top of mine shaft at correct elevations and mark with: "Caution-Raidoactive Material".
- re-spread topsoil and plantlife back over borrow area. Use fertilizer if necessary.
- call in the U.S. NRC for final contamination survey.
- terminate U.S. NRC license.
- site abandonment.

The critical path for all work described above would be that of emplacing the capillary break and the cover material: All other work would be completed within this time.

During all of the above-mentioned work radiological monitoring will be conducted to determine the extent(s) of contamination, to prevent the unnecessary spreading of contamination, to guide the management in charge of the reclamation project, and to evaluate the reclamation project onsite. The work will be overseen by a project manager, a geotechnical engineer and a radiation safety officer to ensure that all specifications are met.

Decommissioning of the mill begins when the mill can not be returned to operational status. Reclamation of the site, generally, begins when it is clear of all structures and foundations. Areas expected to be contaminated will be surveyed and mapped. Grid patterns shall be chosen, dependent upon the location; for example, large areas can be successfully surveyed using a ten-metre grid, provided that all areas are scanned so that small "hot spots" are not overlooked--gamma radiation surveys conducted with portable instruments are the usual method employed. Measurements are noted on the grid intersections. Notations of gamma measurements are usually expressed as fractions, with the ground measurement representing the denominator and the waist-level measurement as the numerator. The waist-level measurements are taken to evaluate the contribution of gamma-emitting radionuclides over the immediate area, as compared to the small area "seen" by the instrument on ground contact. Upon completion of the initial gamma radiation survey, measurements are evaluated and areas of concern delineated. Engineering methods for removal and radiation safety guidelines can then be established. Areas found to be contaminated will then be sampled to determine the depth of contamination in the soil.

Before implementing the afore-mentioned work, further tests will be conducted on the tailings and the proposed borrow area to be used for the main cover material to more accurately assess geotechnical parameters. In addition, because RAMC expects to include a clay layer within the tailings cover, onsite materials such as Mancos shale will be investigated; it is probable that the final cover will consist of a six-inch sand blanket, overlain by six inches of natural soil, six inches of clay, and then natural soil, respectively. With U.S. NRC approval, RAMC expects to conduct these tests in 1986.

As a final remark, RAMC understands the need to stabilize the tailings by consolidation, or compaction. But, as already outlined, it does not know of any reasonable, practicable method to achieve this objective. It may, therefore, be argued--is it really necessary? Would the additional costs involved be justified? What could go wrong if the tailings are not stabilized? What is the worst-case scenario?

The flood protection plan is designed to eliminate any problem associated with the subsequent natural compaction of the tailings--even up to 1000 years. If natural settling does take place, the proposed flood diversion plan will not be impacted because settling will provide additional flood storage volume and thus further attenuate offsite runoff rates. However, if subsequent settling does occur, cracks may be formed in the cover material which may be responsible for releases of radon in excess of the 20 $pCi/m^2 sec$ standard. This is one more reason why the slurrying method is recommended for placing the cover materials. Cover materials so placed will be less brittle than compacted materials; they will be more plastic and less susceptible to "fracturing".

The worst-case scenario may be caused if the probable maximum earthquake event occurred. It may cause the tailings to "liquify" and tend to seek their most stable position: that of a horizontal surface. However, with gentle slopes of 2½% and less, it is very doubtful that such forces will be great enough to overcome internal frictional resistances and the cover material weight. In addition, the integrity of the tailings embankments under this maximum probable earthquake condition was addressed in Section 6.3. Lack of phreatic surfaces, high pseudo-static factors of safety, and the fact that no faults exist beneath the embankments ensure their future stability. Nevertheless, if movement of the tailings does occur, fractures may occur in the cover which may lead to local radon emission rates above the 20 pCi/m^2sec standard--certainly not enough to justify spending needless millions of dollars. That is what the "ongoing maintenance" bond is for. Should any of the above-mentioned problems occur, sufficient funds are available to rectify them.

Referenced Documents for Subsection 5.5-12

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2. Earthfax Engineering, Inc., "Remedial-Action Plan for Groundwater Contamination Control at the Lisbon Uranium Mill", April 1984.
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8. C. E. Israelsen, et al, "Erosion and Sedimentation in Utah: A Guide for Control", Utah Water Research Laboratory, Logan, UT., February 1984.

This page is a reference page used to track documents internally for the Division of Oil, Gas and Mining

Mine Permit Number M0370001 Mine Name Lisbon mine
Operator Rio Algom Mining Corp Date April 23, 1990
TO ______ FROM ______

__CONFIDENTIAL __BOND CLOSURE __LARGE MAPS X EXPANDABLE
__MULTIPUL DOCUMENT TRACKING SHEET __NEW APPROVED NOI
__AMENDMENT _OTHER______

Description YEAR-Record Number

__NOI X Incoming __Outgoing __Internal __Superceded

Operation Plan

__NOI __Incoming __Outgoing __Internal __Superceded

__NOI __Incoming __Outgoing __Internal __Superceded

__NOI __Incoming __Outgoing __Internal __Superceded

__TEXT/ 81/2 X 11 MAP PAGES __11 X 17 MAPS __LARGE MAP

COMMENTS: ______

CC: ______